GEOTECHNICAL REPORT

SR-539 Duffner Ditch – Fish Passage

XL6478, NWR, SR-539 (MP 11.08)

Washington State Department of Transportation
Multimodal Development & Delivery
Construction Division
Geotechnical Office
1655 2nd Avenue SW
Tumwater, WA 98512-6951

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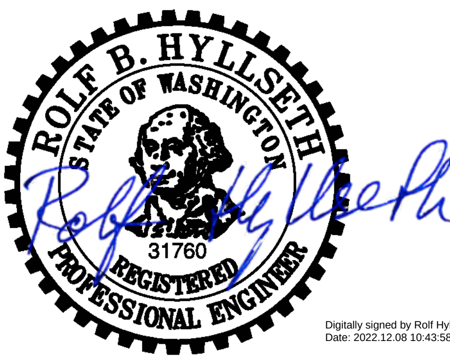

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1 INTRODUCTION

This report presents the results of our geotechnical investigation and contains geotechnical recommendations for design and construction of a precast concrete culvert structure, retaining walls, and embankment modifications at a fish passage improvement site where Duffner Ditch crosses under SR 539 in Lynden, WA. This report was prepared by the Washington State Department of Transportation (WSDOT) Headquarter (HQ) Geotechnical Office for use by the Northwest Region Project Engineer's Office (PEO). When the Plans, Specifications, and Engineering (PS&E) is completed for this project, our office will provide a Summary of Geotechnical Conditions for inclusion in the Special Provisions.

The analyses, conclusions, and recommendations in this report are based upon two exploration borings advanced near the inlet and outlet of the originally planned Duffner Ditch culvert location (about 80 feet south of final). In addition to the explorations, published geologic information for the site vicinity and our experience with similar geologic materials also formed the basis of the analyses, conclusions, and recommendations in this report.

The exploratory borings are assumed to be representative of the subsurface conditions throughout the culvert area. However, actual conditions may differ from those represented by the borings. If during construction, subsurface conditions differ from those described in the explorations, we should be advised immediately so we may reevaluate our recommendations and provide assistance.

2 PROJECT OVERVIEW

This project involves the replacement of an existing culvert along Duffner Ditch. The site is located in the vicinity of milepost 11.08 on SR-539. It is located approximately 0.2 miles south of the intersection of SR-539 and SR 546 in Whatcom County to the west of Lynden, Washington. The overall project area is shown on Figure 1, Site Vicinity.

The project includes the construction of a split box (four-sided), precast, reinforced concrete culvert. The proposed culvert location is shown on Figure 2, Site Exploration Plan. The general structure location and dimensions are summarized in Exhibit 2-1.

EXHIBIT 2-1: PLANNED CULVERT LOCATION AND DIMENSIONS

Culvert Designation	Culvert Approximate Location	Width* (feet)	Length (feet)	Height (feet)
Duffner Ditch	MP 11.08, SR-539	22	95	13.6

NOTES:

*The width is the horizontal interior opening measured parallel to the roadway centerline.

According to the project plans, the Duffner Ditch Culvert will have four wing walls (designated NE, SE, NW, SW). Three walls (NW, SW, and SE) will be concrete retaining walls supported by spread footings. The NE Wing Wall be designed as a Structural Earth (SE) wall. The dimensions of the planned walls are detailed in Exhibit 2-2.

EXHIBIT 2-2: PLANNED CULVERT LOCATION AND DIMENSIONS

Wall Designation	Height* (feet)	Length (feet)	Approximate Footing Width (feet)
NE	13	17.8	30[1]
NW	13	8.0	8.25[2]
SE	13	11.8	8.25[2]
SW	13	8.3	8.25[2]

NOTES:

1. Footing width is equivalent to lowermost grid length which is governed by engineering analysis to prevent damage to the traveled way during a liquefaction event.
2. Minimum footing width based on WSDOT Standard Plan D-10.20-01.

The structure will also have a headwall at each end of the culvert. Embankment slopes will be reconstructed at an inclination of approximately 2 or 2.5 Horizontal to 1 Vertical (2 or 2.5H:1V). We understand the concrete structural design of the proposed four-sided box structure and retaining walls will be provided as a submittal by the contractor during construction, based on the geotechnical recommendations provided in this report.

The culvert is more than 20 feet wide (interior clear span along road centerline) and therefore is required to be designed for seismic hazards per the WSDOT Bridge Design Manual (BDM; WSDOT 2022). Per Section 6-1.2.1 of the WSDOT Geotechnical Design Manual (GDM; WSDOT 2022), the NE retaining wall shall be evaluated for seismic hazards that could cause an abrupt elevation change within the traveled way if wall collapse occurs.

Unless otherwise noted, the vertical datum used for the project is the North American Vertical Datum of 1988 (NAVD88), and the horizontal datum is the North American Datum of 1983, State Plane South (NAD83).

3 SITE INVESTIGATION

The project field exploration program consisted of drilling two test borings at the project location and installing and monitoring field instrumentation. Information obtained during the field exploration program was used to characterize the subsurface conditions at the proposed culvert.

Following exploration, the culvert was relocated about 80 feet north of our northernmost exploration location. Although the final culvert location subsurface conditions may vary somewhat from the explorations, this variation is not anticipated to be significant, based on our experience with geological depositional environment in the Lynden area.

3.1 Existing Data Review

We searched WSDOT and other public records for generally available subsurface data to supplement the site-specific explorations. We discovered the following general subsurface information for each site (generally consistent with site borings):

- Well logs and WSDOT records in the vicinity of the project site generally encountered variable subsurface conditions ranging from mixed sand and silt, to clay.

3.2 Field Reconnaissance

We conducted a geologic site reconnaissance of the existing culvert on June 8, 2021. The goal of our reconnaissance was to identify the following (if present):

- The extent and character of exposed soil units.
- Indications of embankment slope and channel bank instability.
- Indications of instability or long-term settlement of the existing roadway and existing culverts.
- Potential geologic hazards or geotechnically challenging conditions that may impact design or construction of the proposed fish passage project.

Duffner Ditch flows east to west through an existing concrete culvert beneath the SR-539 embankment, as shown on Figure 2. Moving away from the roadway, both drainage channels are confined by agricultural fields and commercial developments to the northeast and northwest, and by residential properties to the southeast and southwest. We observed minimal flow in both channels at the time of our visit. Overhead utility lines are present along the west shoulder of SR-539.

Duffner Ditch is an approximately 8-foot-wide drainage channel that runs parallel to SR 539 along the east side of the roadway embankment. Near MP 11.08, Duffner Ditch flows through an existing, circular, four-foot diameter, concrete culvert under SR-539 (see Exhibit 3-1). The culvert is approximately 50 feet long. Water in the channel was less than 1 foot deep at the culvert inlet during the time of our site visit.

EXHIBIT 3-1: EXISTING DUFFNER DITCH CULVERT INLET ALONG THE EASTERN EDGE OF SR 539.

The roadway embankment and the adjacent property embankment near the culvert inlet appear to be shored up by an existing stone retaining wall (see Exhibit 3-2). The wall has a maximum height of about 5 feet and has signs of erosion along the base. At one location the wall has collapsed. The walls extend from the culvert inlet north about 100 feet, where they transition to approximately 2H:1V embankment slopes on either side of the ditch. The embankment slopes are vegetated with grass.

EXHIBIT 3-2: EXISTING RETAINING WALL ALONG THE WESTERN APPROACH DITCH.

The outlet of the Duffner Ditch culvert opens into an approximately 10-foot-wide channel (see Exhibit 3-3). However, it was difficult to determine the exact ditch width at the time of our site visit as the creek was heavily vegetated with blackberries. The ditch runs in an east to west direction after the culvert outlet. The southern embankment is supported by a stone retaining wall with a maximum height of about 4 feet. The wall continues for at least 100 feet before being covered by blackberries and trees. The northern embankment was obscured by blackberries, however, it appeared to consist of slopes as steep as 2H:1V.

EXHIBIT 3-3: CULVERT OUTLET AND EXISTING RETAINING WALL ALONG SOUTH SIDE OF DITCH.

The roadway embankment and channel slope conditions are described below:

- We measured roughly 2H:1V or flatter roadway embankment slopes along the east and west sides of SR 539 where there were no retaining walls.
- Along these roadway embankment slopes, we observed fill soils generally comprised of sand and fine to coarse gravel.
- There are existing retaining walls near both the inlet and outlet of the culvert.

We did not observe any indications of settlement, or tension cracks along SR 539 However, we did notice erosion along the bottom of the ditch retaining walls on the east side of SR 539, with a portion of one of the walls collapsed on the private property side of the ditch. We did not notice other locations where erosion had resulted in local slumping or wall collapse.

3.3 SUBSURFACE EXPLORATION PROGRAM

The project field exploration program consisted of drilling a total of two test borings designated A-31p-19 and H-1-21, located near each end of the existing culvert, as shown on Figure 2. Boring A-31P-19 was drilled during the preliminary geotechnical scoping phase in June 2019 (XL5949) and boring H-1-21 was drilled during final design in April 2021, to supplement preliminary information obtained at the site. Note that this project was initially tracked under a pre-design Work Order number (MS8328), so this is shown on some of the supporting documents included with this report.

A monitoring well was installed in boring A-31-19, which recorded groundwater data for approximately 12 months between July 2019 and July 2020. The Information obtained during the field exploration program was used to characterize the subsurface conditions at the proposed culvert location.

The test borings were drilled using wet rotary methods and a casing advancer. Standard Penetration Tests (SPTs) were performed in all the test borings. Continuous or back-to-back sampling was performed at each boring location at depths of up to approximately 20 feet below ground surface (bgs). Then, SPTs were generally collected in approximate 5-foot intervals thereafter. Shelby Tube sampling was performed at the H-1-21 boring located in the southwest corner the planned culvert. Details of sampling during drilling are presented on the boring logs in Appendix A.

A WSDOT drill inspector collected soil samples and completed a visual classification of recovered samples in the test borings. Following completion of drilling and sampling, the WSDOT drill crew conducted bail and recharge testing on test boring A-31p-19. Logs of the test borings are included in Appendix A, along with a more detailed description of drilling/sampling techniques used and a legend to aid in the interpretation of the logs. The boring logs provide additional details on the sampling.

3.4 Laboratory Testing

Laboratory testing was performed by the WSDOT Material Laboratory and Haley & Aldrich on selected soil samples for the purposes of classification and development of soil engineering properties. Tests performed included natural moisture content, Atterberg limits, and grain size determination. Laboratory testing was performed in general accordance with appropriate American Society for Testing and Materials (ASTM) and American Association of State Highway and Transportation Officials (AASHTO) test methods. Laboratory test data are provided in Appendix B.

3.5 Field Instrumentation

Upon completion of the A-31p-19, an open standpipe piezometer with a pressure transducer and Level Troll 500 data logger was installed to approximately 21 feet bgs, with a well screen from approximately 10 to 20 feet bgs. No monitoring well was installed during the 2021 exploration phase. The description of the installed piezometer is included on the boring logs in Appendix A.

The piezometer was continuously monitored for a period of at least one year and the results are presented in Appendix C. This piezometer has been decommissioned in accordance with Washington Administrative Code (WAC) 173-160).

4 GEOLOGY AND SUBSURFACE CONDITIONS

4.1 SITE GEOLOGY

As part of this project, we reviewed available geologic data and provided a site-specific geologic map for the planned culvert location based on a 1:100,000 scale geologic map of the area (DNR 2016; Lapen, 2000), as shown on Figure 3, 100K Geology Map. According to Lapen (2000), the project location is generally underlain by a Fraser-age Continental Glacial Outwash, described primarily as well-sorted gravel with local boulders, sandy gravel, and rare sand to silt. However, older but more localized 1:62,500 scale mapping indicates that the outwash unit generally grades to sand near Lynden (Easterbrook, 1976). Surficial Peat is mapped in the region, but the closest mapped Peat is located approximately 5,100 feet southwest of the project site. Based on nearby geologic mapping the outwash is likely underlain by Marine Deltaic Outwash, generally consisting of clay.

4.2 ENGINEERING STRATIGRAPHIC UNITS (ESUS)

Based on soil type and density from site borings, laboratory engineering properties, and geologic origin, we have developed Engineering Stratigraphic Units (ESUs) for the soils encountered at the site.

The site is anticipated to be underlain by Fraser-age Continental Glacial Outwash, likely underlain by Marine Deltaic Outwash at depth. Based on this information and logs of borings drilled at the site, we have classified the site soils into three ESUs, as shown in Exhibit 4-1.

EXHIBIT 4-1: PROJECT-SPECIFIC ESUS

ESU #	Description	Below Ground Surface (feet)*
1	Alluvium: Very loose to loose, moist to wet, sand with varying amounts of silt and gravel, organics may be present.	0-12
2	Continental Glacial Outwash: Medium dense to dense, wet, poorly graded sand with silt to silty sand.	6-50
3	Marine Deltaic Outwash: Soft to medium stiff, moist to wet, fat clay to lean clay.	50-85

NOTES:
*The depths listed here are approximate and relative to existing surface grades at boring locations.

Water well logs and available WSDOT boring logs in the vicinity of the site generally encountered similar sandy/gravelly soils over deeper, interlayered clay and silty sand, confirming that the soil conditions observed in the deeper boring are appropriate to assume for design across the entire project area. The subsurface materials encountered in the boring and water well logs are generally consistent with the mapped geology.

A cross-section of the existing ground surface with subsurface data from the exploration logs is presented on Figure 4, Subsurface Profile. Note that soil descriptions and interfaces shown on the subsurface profiles are interpretive based on corresponding ESUs and should not be assumed to be perfectly representative of actual site conditions.

4.3 SURFACE WATER AND GROUNDWATER

4.3.1 SURFACE WATER

The existing stream (Duffner Ditch) is the primary source of natural surface water in the immediate vicinity of the project site.

4.3.2 GROUNDWATER

We anticipate the groundwater levels at the sites are primarily controlled by seasonal groundwater variations but could also be affected by fluctuating water levels of nearby creeks. The piezometer installed in boring A-31p-19 was monitored continuously from installation through July 2021 (about one year). The groundwater level during this period generally fluctuated about 8 feet, between 5- and 13-foot depth bgs (Elevation 82.5 and 90.8).

Based on these measurements, the groundwater table varies throughout the year and generally correlates with the surface water the surrounding ditches and creeks. The maximum groundwater elevation during the monitoring period for the project site is shown in Exhibit 4-2. A plot of the groundwater level readings from the piezometer/transducer (along with rainfall data) is presented in Appendix C.

EXHIBIT 4-2: GROUNDWATER DATA SUMMARY

Boring	Maximum Groundwater Elevation (feet)*	Maximum Groundwater Measurement Date
A-31p-19	90.8	2/1/2020

NOTES:
* Elevation NAVD 88

4.4 POTENTIAL VARIANCE

The subsurface interpretation and engineering analyses are based on the field exploration and laboratory testing program described previously. These interpretations are specific to the locations and depths noted on the boring logs (Appendix A) and may not be applicable to all areas of the site. No number of explorations can precisely predict the characteristics, quality, or distribution of subsurface conditions. Potential variation from what is shown on the boring logs includes, but is not limited to:

- The conditions between and below explorations may be different.
- The passage of time or intervening causes (both natural and man-made) may result in changes to site and subsurface conditions.

- Groundwater levels at the site fluctuate seasonally due to precipitation and creek levels and may be higher or lower than measured during our monitoring period.
- SPT N-values in gravelly and cobble-rich soils may be unrealistic. Actual soil density may be lower than estimated from the N-values if the test was performed on a piece of gravel or cobble.
- Although not specifically encountered in the borings, boulders may be present within ESU 2.

If conditions different from those described herein are encountered during construction, we should review our interpretation and reconsider our geotechnical recommendations presented herein.

5 GEOLOGIC HAZARDS

5.1 Scour

According to information provided to us by the PEO, the anticipated scour depth at the inlet of the Duffner Ditch culvert is approximately 4.4 and 5.1 feet for the 100- and 500-year event, respectively. There is no anticipated scour at the culvert outlet.

5.2 Seismic Hazards

As noted in Section 2 of this report, seismic design is required for the culvert and the wing walls that are greater than 10 feet high and could cause an abrupt elevation change within the traveled roadway above if wall collapse occurs (see Section 6.1).

We evaluated potential seismic shaking at the site in accordance with the GDM, which considers the design earthquake to be seismic shaking having a 7 percent probability of exceedance in 75 years (approximately 1,000-year return period). Our evaluation used data obtained from WSDOT’s Spectra ground motion software and U.S. Geological Survey (USGS) Unified Hazard Tool Dynamic Conterminous (USGS 2019).

Based on this, the expected peak bedrock acceleration having a 7 percent probability of exceedance in 75 years is 0.286g. This value represents the peak acceleration on bedrock beneath the site and does not account for ground motion amplification due to site-specific effects. The peak ground acceleration (PGA) is determined by applying a site class factor to the peak bedrock acceleration (see Section 6.2). Based on the deaggregation of the site seismic hazard, the mean magnitude is 6.9 and distance to rupture is approximately 59 kilometers (km). The modal magnitude is 7.1 with a distance to rupture of approximately 69 km.

5.2.1 Fault Rupture

The potential impacts of fault rupture include abrupt, large, differential ground movements and associated damage to structures that might straddle a fault, such as a bridge abutment or retaining wall. The USGS maintains information on faults and associated folds in the United States that are believed to be sources of magnitude 6 or higher earthquakes during the Quaternary period (USGS, 2019). Based on our review of the USGS Interactive Fault Map, the closest fault to the site, is the Dayton Harbor Fault Scarp, which is about 7.3 miles from the site. This is depicted on Figure 5, Fault Map. Due to the significant distance of the nearest faults from the project sites (greater than the 6-mile AASHTO bridge design separation criteria), the risk of fault rupture effects at the culvert site from the nearby faults is low, in our opinion.

5.2.2 LIQUEFACTION POTENTIAL

Soil liquefaction is a phenomenon whereby saturated soil deposits temporarily lose strength and behave as a viscous fluid in response to cyclic loading. Soil types considered at the highest risk of liquefaction during a seismic event are saturated and are loose to medium dense granular soils. We performed an analysis to determine the liquefaction susceptibility of the soils that were encountered at each of the drilled borings, using SPT correlations by Idriss and Boulanger (2008). Based on the results of these analyses, the saturated portions of ESU 1, 2 and 3 are considered liquefiable.

Post-liquefaction (or reconsolidation) settlement occurs because granular liquefiable soils tend to get redistributed and become denser after the earthquake and after excess porewater pressure dissipates. The ground surface settlement is not typically uniform across an area and can result in significant differential settlement. Using the analysis methodology by Idriss and Boulanger (2008), we anticipate overall liquefaction-induced settlement up to 5.0 inches across the culvert site. We anticipate that the liquefaction induced differential settlement may be on the order of 2.0 inches over a distance of 50 feet. This settlement occurs primarily within the shallow cohesionless ESU 1 and ESU 2 layers that were encountered at depths between 5 feet and 25 feet bgs in borings A-31p-19 and H-1-21, but also within the deeper, medium dense silty sand layers that were encountered at depths between 75 feet and 80 feet bgs in borings H-1-21 (ESU 3).

5.2.3 RESIDUAL SHEAR STRENGTH AND CYCLIC SOFTENING

For soil samples where the factor of safety (FS) against liquefaction is less than 1.2 in our liquefaction analysis, we developed residual friction angles in accordance with Idriss and Boulanger (2008) to characterize the liquefied soil strength for loading conditions after an earthquake event (i.e., post-seismic conditions). We used this reduced residual soil shear strength in our culvert/wall bearing and global stability analyses where it would likely affect our culvert/wall bearing and global stability analyses (ESU 1 and ESU 2 from 5 to 25 feet).

Cyclic softening (shear strength weakening) of fine-grained soils that are not susceptible to liquefaction, such as silts and clays with medium to high plasticity, depends on the sensitivity of the soil. Based on the results of Atterberg Limit tests, the Fat Clay layers within ESU 3 are medium sensitive and therefore may see a strength reduction of 10 to 15 percent, per WSDOT GDM Section 6-4.3.1. However, because of the significant depth to these fine-grained soils, this potentially reduced soil strength condition will not affect the stability of the planned near-surface structures, in our opinion.

6 GEOTECHNICAL ANALYSIS AND RECOMMENDATIONS

The following sections describe the engineering analyses and geotechnical engineering recommendations for the proposed culvert and associated wing walls. Our recommendations are based on the project requirements and the project plans prepared by the PEO, our discussions with the PEO, and our interpretation of the subsurface conditions described herein.

We have prepared our design recommendations considering the project configuration as described herein. If the PEO develops additional or revised information about final foundation and wall configurations or other factors, the recommendations presented herein may need to be revised. The Geotechnical Office must be made aware of the revised or additional information so that we can evaluate our recommendations for applicability.

For purposes of our analyses, it was necessary for us to assume that the results of the subsurface explorations, as described in Sections 3 and 4 of this report, are representative of conditions at the respective project sites. However, subsurface conditions should be expected to vary (see Section 4.4). We may need to revise our recommendations if different conditions are encountered during construction.

6.1 DESIGN CRITERIA AND GENERAL CONSIDERATIONS

This Project will be designed in accordance with the GDM, the BDM, and the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO, 2020). Based on these documents, the following specific design criteria and conditions/assumptions were considered when performing our analyses:

- Structures requiring seismic design shall meet the Safety Evaluation Earthquake performance level objectives of no-collapse, as described in GDM Section 6-1.2.1.
- Seismic design is required for the box culvert since it is more than 20 feet wide (inside clear span dimension) along the centerline of the roadway, per section 8.3.1 of the BDM.
- Seismic design to prevent collapse is required for the NE Wing Wall because it is taller than 10 feet at its highest point and supports a traveled roadway that could be subject to an abrupt change in elevation in the event of wall failure (reference GDM Section 6-1.2.1).
- Seismic design is required for the NW, SW, and SE Wing Walls but does not have to prevent collapse in the extreme design case, since these walls are far enough from the future planned roadway or can be oriented to where potential collapse would not impact the travelled way or compromise the general life safety or the public.
- Liquefaction induced site settlement is anticipated to be up to 5 inches, with differential settlement estimated to be up to 2 inches (over 50 feet).
- Wall and culvert foundation elements shall be designed for the effects of potential 100- and 500-year event scour levels (BDM Section 8.3.3.D).

- Due to the expected bearing soil strength loss during soil liquefaction, we recommend that the planned NE Wing Wall be designed as a SE wall to prevent seismic collapse that may impact the traveled way. This can be accomplished using WSDOT Standard Plans D03.10-1 and D03.09-0 or using a proprietary wall design found in Appendix 15-D of the GDM.
- The planned NW, SW, and SE Wing Walls may be designed as conventional concrete retaining walls, as long as adequate distance and wall orientation can be provided to prevent a potential extreme event collapse will not impact the future travelled way.
- Although temporary sloping is the responsibility of the Contractor and will depend on field conditions encountered during construction, we have assumed a conservative 1.5H:1V temporary slope inclination (flattest standard code-based slope condition), where applicable in our analyses. If needed for PEO planning purposes and to estimate quantities prior to construction, we recommend assuming this temporary sloping condition will be feasible or consider the need for structural shoring. This is for planning purposes only. The actual safe slope should be determined during construction and is the responsibility of the contractor.

Additional analysis-specific criteria are referenced in subsequent sections.

6.2 SEISMIC DESIGN PARAMETERS

The ground shaking hazard can be defined in general terms using appropriate acceleration response spectra and site coefficients, or by using a site-specific procedure. For the general procedure, the spectral response parameters are determined using the 2014 Seismic Hazard Maps produced by the USGS depicting probabilistic ground motion and spectral response for 7 percent probability of exceedance in 75 years.

Based on AASHTO Article 3.10.3.1, we classified the site soils as Site Class E. Therefore, the general procedure can be followed. In accordance with GDM Section 6-3, the coefficients provided in Exhibit 6-1 should be used. Design parameters for foundation springs and structure racking analyses are also provided, in general accordance with the methods outlined in Section 6-5.1.1 of the GDM.

EXHIBIT 6-1: SEISMIC DESIGN PARAMETERS

Parameter	Recommended Value
Site Class Based on Soil Conditions	Site Class = E
Mean Magnitude	M = 6.9
Modal Magnitude	M = 7.1
Peak Ground Acceleration (PGA) Coefficient of Class B Rock	PGA = 0.286g
0.2-Second Period Spectral Acceleration Coefficient on Class B Rock	S_s = 0.646g
1.0-Second Period Spectral Acceleration Coefficient on Class B Rock	S_1 = 0.188g
Site Coefficient for the Peak Ground Acceleration Coefficient	F_{pga} = 1.641
Site Coefficient for 0.2-Second Period Spectral Acceleration	F_a = 1.466
Site coefficient for 1.0-Second Period Spectral Acceleration	F_v = 3.407
Effective Peak Ground Acceleration Coefficient (g)	$A_s = F_{pga}$*(PGA) = 0.470g
Design Earthquake Response Spectral Acceleration Coefficient at 0.2-Second Period	$S_{DS} = F_a$*S_s = 0.947g
Design Earthquake Response Spectral Acceleration Coefficient at 1.0-Second Period	$S_{D1} = F_v$*S_1 = 0.641g
Dynamic Shear Modulus of Foundation Soils	G = 2,000 ksf
Poisson's Ratio of Foundation Soils	v = 0.30
Racking Deformation	ΔS = 0.57 in
Free Field Ground Shear Strain	γ_{max} = 0.474 %

6.3 PRECAST CONCRETE CULVERT

We understand that a four-sided precast concrete box culvert will be used at the planned fish passage location. The approximate dimensions of culvert are presented in Section 2 of this report. Based on a design scour depth of 4.4 feet and a creek thalweg of 89 feet provided by the PEO, we recommend that the bottom of culvert design elevation not be higher than 82.6 feet (NAVD 88). This includes a 2-foot embedment depth below the design scour elevation, in accordance with GDM Section 15-4.5.

6.3.1 BEARING RESISTANCE

We performed bearing resistance calculations to assess the adequacy of underlying soils to support each box culvert assembly segment on the prepared subgrade, assuming these are free to move relative to each other. We assumed the precast box culvert segments will be as long as the overall culvert outside width and a typical 5 to 10 feet wide (along the culvert alignment). According to AASHTO Article 10.6.3.1.2b, if local or punching shear failure is possible, the nominal bearing resistance shall be estimated using reduced shear parameters. The estimated nominal (unfactored) bearing resistances are presented in Exhibit 6-2. The resistance factors presented in Exhibit 6-3 should be applied to the nominal bearing resistances presented in Exhibit 6-2 in order to determine the factored bearing resistances.

EXHIBIT 6-2: NOMINAL BEARING RESISTANCE FOR CULVERT SECTIONS

Precast Section Length (feet)	Precast Section Width (feet)	Unfactored Strength and Extreme Event (EQ Loading) Limit State Bearing Resistance (ksf) [1]	Unfactored Extreme Event (Post-Seismic Liquefaction) Limit State Bearing Resistance (ksf) [2]	Unfactored Service Limit State Bearing Resistance* (ksf) [3]
22	5	3.7	1.9	5.8
22	6	3.9	2.0	5.3
22	7	4.1	2.0	4.9
22	8	4.3	2.0	4.6
22	9	4.5	2.1	4.4
22	10	4.7	2.1	4.2

NOTES:

1. Extreme Event during EQ shaking – Design Bearing Soil Pressure should include dynamic lateral loading and wall inertial forces.
2. Post-EQ Extreme Event (soil liquefaction) – Design Bearing Soil Pressure should be based on Service Limit loading conditions.
3. Based on 1 inch settlement.

Excavations for the base of the culvert are anticipated to reveal either ESU 1 or ESU 2, which are both considered suitable bearing material. Due to the potential for these materials to liquefy and the need to improve culvert stability/performance during a design earthquake event, we recommend overexcavating the upper 2.5 feet of the exposed native subgrade soil and replacing this with:

- 6 inches of Culvert Bedding Material per Section 9-03.12(3) of the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction (WSS) (WSDOT 2023), or WSS 9-03.1(4) C AASHTO Grading 57; overlying
- 2.0 feet of Permeable Ballast per WSS 9-03.9(2) or quarry spalls per WSS 9-13.1(5).

This overexcavation and replacement should extend approximately 1 foot beyond the footprint of the culvert.

This overexcavation will improve the subgrade working conditions and also provide a uniform surface to place the structure segments. To reduce the potential for migration of soil through the rock, a Class B (moderate survivability) Geotextile for Underground Drainage should be installed between the native soils and the Permeable Ballast or Quarry Spalls in accordance with Section 9-33.2(1) of the WSS.

For the extreme event limit state, we estimate that up to approximately 5 inches of liquefaction-induced settlement may occur in certain areas across the culvert footprint. The resulting differential seismic settlement is anticipated to vary across the site up to 2 inches per 50 feet of culvert length/width. The proposed culvert should be designed to accommodate these estimated differential settlements due to liquefaction (meeting the no-collapse design objective).

6.3.2 RESISTANCE FACTORS

Resistance factors for bearing resistance at each respective limit state are presented in Exhibit 6-3. These factors were determined using the GDM Chapter 8 and AASHTO Article 10.5.

EXHIBIT 6-3: SPREAD FOOTING RESISTANCE FACTORS FOR CULVERTS AND WING WALLS

Resistance Factor, ϕ		
Strength	Service	Extreme Event
0.45	1	0.9

6.3.3 LATERAL DESIGN PARAMETERS

The side walls of the precast concrete culvert should be designed to resist lateral earth pressures under strength and extreme event limit state conditions, in accordance with AASHTO and BDM. We have assumed that the culvert walls will be backfilled with Gravel Backfill for Walls per Section 9-03.12(2) of the WSS. The recommended lateral geotechnical design parameters for the planned four-sided box culvert are presented in Exhibit 6-4.

EXHIBIT 6-4: LATERAL DESIGN PARAMETERS FOR FOUR-SIDED BOX CULVERTS AND WING WALLS

Parameter	Design Value
Backfill Moist Unit Weight (γ)	130 pcf
Backfill Buoyant Unit Weight (γ')	67.6 pcf
Wall Backfill Soil Friction Angle	38^o
Coefficient of Sliding (tan ϕ_f)	0.6
At Rest Earth Pressure Coefficient (K_a) – Level Backfill	0.24
At Rest Earth Pressure Coefficient (K_o) – Level Backfill [1]	0.38
Passive Earth Pressure Coefficient (K_p) – Level Backfill	4.2
Seismic Active Earth Pressure Coefficient (K_{ae}) – Level Backfill [2]	0.37
Seismic Passive Earth Pressure Coefficient (K_{pe}) – Level Backfill [2]	3.71

NOTES:

1. Assuming unyielding wall condition
2. The seismic earth pressure coefficients are cumulative values including the static earth pressure component.

pcf = pounds per cubic foot

The side walls for a precast culvert should be designed for at-rest earth pressures, using the Ko value presented in Exhibit 6-5.

Below the groundwater table, the buoyant unit weight should be used and a hydrostatic force should be added to the submerged soil lateral earth pressure.

A resistance factor of 0.9 and a footing type coefficient of 0.8 should be used in conjunction with the earth pressure coefficient above for sliding resistance analysis of precast structures placed on prepared backfill material. For passive earth pressure to resist sliding, we recommend using a resistance factor of 0.5.

6.4 Concrete Retaining Walls (NW, SW, and SE Wing Walls)

Contractor designed, precast concrete wing walls will be used on both sides of the culvert outlet (at west end) and on the south side of the culvert inlet (at east end), to retain road embankment soil and provide a transition from the roadway down to the toe of the creek/channel side slope. Our current understanding of the proposed wing wall lengths and heights is summarized in Exhibit 2-2, and the wingwall bearing elevations are provided in exhibit 6-5. Based on the planned approximate footing bearing elevation, we understand that the walls will be supported directly on either ESU 1 or ESU 2 materials.

Exhibit 6-5: Planned NW, SW, and SE Wing Wall Bearing Elevations

Wing Wall	Approximate Bearing Elevation (feet) [1]
Northwest (outlet)	82
Southwest (outlet)	82
Southeast (inlet)	83 [2]

NOTES:
1. Elevation Reference: NAVD 88
2. Elevation based on scour requirements

6.4.1 Global Stability

We performed a global stability analysis to evaluate standard footing-supported wing walls under static conditions. These were based on the dimensions described in Exhibit 2-2 (per project wall site data package, updated September 27, 2022).

We assumed that the pre-cast concrete cantilevered walls will be backfilled with Gravel Backfill for Walls per Section 9-03.12(2) of the WSS, with a friction angle of 38 degrees. We further assumed the backfill prism will be a triangular wedge behind the wall, above a 1.5H:1V temporary excavation slope.

Our analyses indicate that the planned walls meet the FS requirements for global stability specified in the GDM under static conditions. As previously discussed, these wing walls may be susceptible to collapse in the extreme design case as the underlying soils liquefy. However, this is acceptable as long as adequate distance and wall orientation can be provided to prevent a potential collapse from impacting the future travelled way. We understand that the NW, SW, and SE Wing Walls have been reoriented as needed to meet these criteria.

6.4.2 Bearing Resistance for Wing Walls

We performed an analysis to determine the nominal bearing resistance for the wing wall footings under the strength, service, and extreme limit state conditions. The wall length was assumed to be 8 feet for the NW and SW walls, and 12 feet for the SE wall (per project wall site data package), and we performed the analysis with footing widths ranging from 7 to 8 feet for an estimated 13- to 14-ft wall height (based on WSDOT Standard Plan D-10.25-01). If the final wall foundation is narrower than this, additional bearing analyses may be necessary (contact HQ Geotechnical Office).

The recommended bearing resistances for each respective limit state condition is presented in Exhibit 6-6 and should be used in conjunction with the resistance factors in Exhibit 6-3 to determine factored resistances.

Exhibit 6-6: Nominal Bearing Resistance for Wing Walls

Wall	Precast Section Width (feet)	Unfactored Strength and Extreme Event (EQ Loading) Limit State Bearing Resistance (ksf) [1]	Unfactored Extreme Event (Post-Seismic Liquefaction) Limit State Bearing Resistance (ksf) [2]	Unfactored Service Limit State Bearing Resistance* (ksf) [3]
NW & SW	7	20.3	2.5	7.0
	8	21.2	2.5	6.6
SE	7	19.6	2.5	6.2
	8	20.4	2.5	5.9

NOTES:

1. Extreme Event during EQ shaking – Design Bearing Soil Pressure should include dynamic lateral loading and wall inertial forces.
2. Post-EQ Extreme Event (liquefied bearing soils) – Design Bearing Soil Pressure should be based on Service Limit loading conditions.
3. Based on 1 inch settlement.

As previously noted, we estimate that up to approximately 5.0 inches of liquefaction-induced settlement may occur across the site in the extreme event design limit state, including the wing wall locations.

Note that the nominal bearing resistance recommended in the post-seismic extreme event (liquefied bearing soils) are based on the inclusion of a non-liquefiable, 2-ft thick, reinforced Gravel Borrow base pad below the wall foundation, extending 5 feet beyond the wall foundation edges. The reinforcement should consist of one geogrid reinforcing layer with a minimum long-term tensile strength of 5,200 pounds per foot (lbs/ft).

6.4.3 Lateral Design Parameters

Lateral resistance against sliding for concrete retaining walls is developed by passive earth pressures on the sides of footings and by friction along bearing surfaces. We recommend using the parameters in Exhibit 6-4 for determining lateral resistance against sliding. If wing walls are restrained from rotation prior to being backfilled, then they should be designed for at-rest earth pressures (Ko). If wing walls are allowed to rotate at least 0.1 percent of the wall height when backfilled, then they may be designed for active earth pressures (Ka).

Assuming level backfill, the upper 1 foot below anticipated maximum scour depth should be ignored when determining passive earth pressure. If the wall is supported on a 2H:1V or flatter embankment slope, the upper 2 feet should be neglected for passive pressure design.

A resistance factor of 0.5 should be applied to the nominal passive earth pressure. A resistance factor of 0.9 for pre-cast structures and 0.8 for cast-in-place structures should be applied to the nominal resistance determined from friction on bearing soil surfaces. A footing type coefficient of 0.8 and 1.0 for pre-cast and cast-in-place structures, respectively, should be applied to the nominal resistance against sliding from friction on bearing surfaces.

6.4.4 Wall Drainage

Our recommendations do not consider hydrostatic (water) pressure acting on the wing walls. Therefore, back-of-wall drainage systems should be installed behind the walls, consistent with the requirements of Section 7.5.11 of the BDM. If drain lines cannot be diverted to a local stormwater conveyance system, weep holes may be used, provided potential risk of soil slope erosion is addressed.

6.5 Structural Earth Wall (NE Wing Wall)

Due to the expected bearing soil strength loss during soil liquefaction, we recommend that the planned NE Wing Wall be designed as a structural earth wall to prevent seismic collapse that may impact the traveled way. This can be accomplished using WSDOT Standard Plans D03.09-0 / D03.10-1 and WSS Section 6-14. Alternatively, a geosynthetic reinforcement based proprietary wall system found in Appendix 15-D of the GDM may also be used, in conjunction with WSS Section 6-13 (excluding steel reinforced systems).

For this site, the proprietary/standard plan walls should be maximum 14 feet high and founded at elevation 82.5 feet, and should include additional longer base reinforcement as described in Section 6.5.1.

The approximate planned NE Wing Wall dimensions are presented in Exhibit 6-7. The bottom of the wall at the culvert inlet must be no higher than elevation 82.6 feet in order to account for scour. As such, the planned walls will likely bear on ESU 2.

EXHIBIT 6-7: PLANNED NE WING WALL DIMENSIONS

Wing Wall	Length (feet)	Maximum Height (feet)	Approximate Bearing Elevation (feet) [1]
Northeast (inlet)	18	14	82.5

NOTES:

1. 2-ft embedment below anticipated scour depth of 84.5 ft.
2. Elevation Reference: NAVD 88

As described subsequently, we have evaluated the overall global stability and have provided modified geometric and subgrade reinforcement requirements for a structural earth wall system that satisfies minimum overall structure stability requirements (including compound analysis). However, proprietary structural earth walls should also be designed and evaluated for internal stability by the proprietary wall supplier, in general accordance with the specifications provided in WSS 6-13 – Structural Earth Walls and our recommendations in the following sections.

6.5.1 STRUCTURAL EARTH WALL GLOBAL STABILITY

For design feasibility purposes, we performed a global stability analysis of the planned structural earth wall using limit equilibrium methods and the SLIDE computer software. Our analyses indicate that the planned structural earth wall meet the factor of safety requirements for global stability specified in the GDM, if reinforced with 30-ft long and stronger than typical reinforcement within the bottom 3 feet (for the extreme event seismic and post-seismic liquefied soil design cases). We made the following key assumptions in our analysis and design, which should be incorporated into the plans and specifications for this project:

- Soft or deleterious material encountered below the structural earth wall subgrade elevation during construction will be removed and replaced with a bearing prism of Permeable Ballast per WSS 9-03.9(2) or Quarry Spalls per WSS 9-13.1(5).
- The three additional reinforcing layers within the bottom 3 feet of the structural earth wall should be a minimum 30 feet long and spaced a minimum 3 inches apart from the main structural earth wall reinforcement. This longer reinforcement should have a minimum long-term tensile strength of 5,200 pounds per foot (lbs/ft). These minimum requirements meet the required factors of safety of 1.5 and 1.1 for the Strength Limit and Extreme states.
- For the main structural earth wall (14-ft high), the reinforcing layers will have a minimum length of 10 feet or 70 percent of the wall height, whichever is greater, and will be spaced a maximum 16 inches apart. The reinforcement will have a minimum long-term tensile strength of 1,600 lbs/ft. These minimum requirements meet the required factors of safety of 1.5 and 1.1 for the Strength Limit and Extreme Event Limit states.
- If greater reinforcement spacing and/or weaker reinforcement strength is selected by the Contractor for internal stability, then the structural earth wall needs to be reevaluated for compound and external global stability.

6.5.2 Structural Earth Wall Design Parameters

The structural earth wall design should be based on the soil parameters presented in Exhibit 6-8.

Exhibit 6-8: Structural Earth Wall Design Parameters

Material	Unit Weight (pcf)	Friction Angle (degrees)
Reinforced Zone Fill (Gravel Borrow for Structural Earth Wall)	130	38
Retained Soil/Backfill (Common Borrow)	125	36
Foundation Soil (ESU 2)	120	31

The Reinforced Zone Fill behind the Structural Earth wall facing should meet the specifications provided in WSS 9-03.14(4) – Gravel Borrow for Structural Earth Wall. The Retained Backfill should meet the specifications provided in WSS 9-03.14(3) Common Borrow. The Foundation Soil should be competent ESU 2 soils evaluated and approved by a WSDOT inspector.

For the seismic evaluation of the structural earth wall, the designer may assume an allowable displacement of 3 inches during seismic shaking. The determination of a horizontal seismic coefficient, k_h, should be based on the PGA adjusted for site class (A_s from Exhibit 6-1 in this geotechnical report). The vertical acceleration coefficient, k_v, may be ignored.

The upper portion of the structural earth soil reinforcing (roughly 11-ft high; above Elevation 86.0 ft) should be a minimum 11 feet long and spaced a maximum 16 inches apart, to meet the strength and extreme design event requirements. The bottom three reinforcing layers of the structural earth wall must be a minimum 30 feet long, spaced a maximum 12 inches apart, and have a minimum long-term design strength of 5,200 lbs/ft.

The structural earth wall foundation design should be based on the minimum embedment depths shown in Exhibit 6-9.

Exhibit 6-9: SE Wall Minimum Foundation Embedment

Exposed Wall Height	Level Toe	Toe Slope at 2H:1V
≤ 4 feet	1 foot	2 feet
4 to ≤10 feet	1.5 feet	3 feet
10+ feet	2 feet	4 feet

The base of the SE wall facing blocks should be supported by a concrete leveling pad. The leveling pad should be cured a minimum of 12 hours and have a minimum compressive strength of 1,500 pounds per square inch (psi) before placement of facing panels or concrete blocks. The leveling pad should extend at least 6 inches in front of and behind the SE wall facing blocks.

6.5.3 Bearing Resistance

As discussed above, the SE wall will generally be supported directly by ESU 2 materials. We performed an analysis to determine the nominal bearing resistance that may be used to design the SE wall under the strength, service, and extreme event limit state conditions for the bearing conditions noted above. The recommended nominal (unfactored) bearing resistances for each respective limit state condition are presented in Exhibit 6-9 and should be used in conjunction with the resistance factors in Exhibit 6-10 to determine factored resistances. For the Extreme Limit state, we assumed a reduced shear strength for liquefied bearing soil assuming punching shear failure may occur (per AASHTO Article 10.6.3.1.2b), but also incorporated the additional support provided by the longer base reinforcement within the lower SE wall.

Exhibit 6-9: Nominal SE Wall Bearing Resistance

Approximate Bearing Elevation (feet)	Reinforcement Length (feet)	Unfactored Strength Limit State Bearing Resistance (ksf)	Unfactored Extreme Event Limit State Bearing Resistance [1] (ksf)	Unfactored Service Limit State Bearing Resistance [2] (ksf)
86.0	11	N/A	3.1	N/A
82.6	30	8.3	N/A	3.2

NOTES:

1. Controlled by post-seismic liquefied bearing soil condition below longer reinforcement wall base. Design Bearing Soil Pressure should be based on Service Limit loading conditions.
2. Based on 1 inch settlement.

ksf = kips per square foot

N/A = Not Applicable

Exhibit 6-10: SE Wall Bearing Resistance Factors

Resistance Factor, ϕ		
Strength	Service	Extreme Event
0.65	1	0.9

At the service limit state, we estimate that the total static settlement of the planned SE wall to be 1.0 inch or less, with an estimated differential settlement of less than 0.5 inches over the wall length.

As previously noted, we estimate that up to approximately 5 inches of liquefaction-induced settlement may occur across the site in the extreme event design limit state, including the SE wall location. The longer/stronger lower reinforcement within the bottom of the SE wall will help prevent seismic collapse and an abrupt surface elevation change within the traveled way. However, the outer portion of the wall (along the shoulder) may experience some settlement due to bearing failure on liquefied soil in the Extreme Event seismic design case. While the amount of such liquefaction-induced settlement is not possible to predict, it is expected that the SE wall reinforcement will constrain the wall movement, resulting in a relatively gradual surface settlement.

6.5.4 Sliding Resistance

Lateral loads on SE walls can be resisted by passive earth pressures on the sides of footings and by friction on bearing surfaces. We recommend using the coefficient of sliding resistance and passive earth pressure coefficients presented in Exhibit 6-4 to determine resistance against sliding for the portions of the wall bearing on ESU 2 or stabilization material. We recommend using a resistance factor of 1.0 and 0.5, respectively, with the sliding and passive earth coefficients listed in Exhibit 6-4.

6.5.5 Wall Drainage

The above design parameters have been provided assuming that back-of-wall drains will be installed to prevent hydrostatic pressures above the groundwater table. Back of wall drainage should be designed per section 15-4.13 of the GDM.

6.6 Headwalls

Based on the project plans, we understand that relatively short headwalls will be used at the inlet and outlet sides of each culvert. If the headwalls are designed to retain embankment fill, then they are acting as retaining walls and need to be designed for lateral earth pressures. Assuming the headwalls will be rigidly attached to the culvert structure, they will need to be designed under strength limit state conditions to resist at-rest lateral earth pressures for unyielding wall conditions.

For lateral load analysis of the headwalls, the geotechnical parameters in Exhibit 6-11 should be used for design. We have assumed that the headwalls will be backfilled with Select Borrow per Section 9-03.14(2) of the WSS.

Exhibit 6-11: Lateral Design Parameters for Culvert Headwalls

Parameter	Recommended Value
Backfill Moist Unit Weight (γ)	130 pcf
Wall Backfill Soil Friction Angle	38^{o}
At Rest Earth Pressure Coefficient (K_o)[1]	0.38
Seismic Active Earth Pressure Coefficient (K_{ae})[2]	0.37

NOTES:

1. Assuming unyielding wall condition
2. The seismic earth pressure coefficients are cumulative values including the static earth pressure component.

pcf = pounds per cubic foot

6.7 Earthwork

A large amount of earthwork will be performed for this Project. The following sections describe recommendations for re-use of excavated material and considerations for temporary slopes and shoring. Our explorations performed for the Project may not be sufficient for design of temporary slopes and shoring. It is the responsibility of the Contractor to conduct additional explorations if needed for the design of their temporary works.

6.7.1 Embankment and Culvert/Wall Backfill Requirements

The contractor will need to design the temporary cut slopes for the excavation, but for planning purposes and to estimate quantities, 1.5H:1V excavation slopes should be assumed by the Project Office. Once the cast-in-place concrete culvert and retaining walls are placed, the contractor should backfill the buried structures in accordance with 2-09.3(1)E of the WSS.

The backfill material directly behind the retaining walls shall be Gravel Backfill for Walls, which shall be placed as shown in WSDOT Standard Plan D-4 Condition B. Backfill material beyond the wall backfill zone can be either Gravel Backfill for Walls (WSS Section 9-03.12(2)) or Gravel Borrow (WSS Section 9-03.14(1)).

Backfill along the side of the cast-in-place concrete culvert should be either Gravel Backfill for Walls or Gravel Borrow.

Above the top elevation of the cast-in-place concrete culvert and the retaining walls, Select Borrow may be used (WSS Section 9-03.14(2)). If the Contractor stockpiles the material from the existing embankment's structure excavation, that material can be used as Select Borrow provided it meets the moisture requirements for compaction and material requirements identified in the WSS. Moisture conditioning may be necessary depending on the moisture content of the material when excavated and stockpiled. If construction will occur during wet weather, Gravel Borrow is recommended rather than reusing the existing embankment material.

Provided embankments are constructed in accordance with the WSS and the above recommendations, all embankment slopes constructed as part of the Project will have an acceptable factor of safety against global failure during static conditions. Embankments are not designed for seismic conditions.

6.7.2 Permanent Cut Slopes

We recommend designing permanent cut slopes for a maximum slope angle of 2H:1V. Until a layer of vegetation is established, the upper 1 to 2 feet below the surface of the slope may be only marginally stable. We recommend that measures be taken to control erosion on new permanent slopes. Such measures should include both short-term and long-term strategies for erosion control. The design of these erosion control measures will be performed by others.

7 GEOTECHNICAL CONSTRUCTION RECOMMENDATIONS

The Project will be constructed per the WSS. We have developed construction considerations for the Project to assist in preparation of Special Provisions and to identify key geotechnical issues that should be prepared for and observed during construction.

Our recommendations are not intended to dictate methods or sequences used by contractors. Prospective contractors must undertake their own independent review and evaluation of the subsurface data to arrive at decisions concerning the planning of the work; the selection of equipment, means and methods, techniques, and sequences of construction; establishment of safety precautions; and evaluation of the influence of construction on adjacent sites.

7.1 Culvert Subgrade Preparation

Although the bottom of excavation is expected to be into medium dense ESU 2 material suitable for culvert and wing wall support, the excavation will likely be below the natural groundwater level and encounter loose granular soil. Therefore, the Contractor should be prepared for challenging excavation conditions and should use excavation, fill placement, and compaction techniques appropriate for the potentially saturated soil conditions anticipated.

A WSDOT geotechnical inspector should review and approve all culvert and wall foundation subgrades, including potential overexcavation and placement of aggregate base material and geotextile separation fabric.

7.2 Temporary Slopes and Shoring

Temporary slopes and/or shoring will be necessary to construct various elements included in this Project. Temporary slopes and shoring are the responsibility of the Contractor, who is solely responsible for site safety. The Contractor will determine the appropriate measures to ensure that all excavation work is in compliance with local, state, and federal safety codes, and in accordance with the requirements in the GDM. WAC Chapter 296-155 contains specific requirements for trenches and temporary slopes, as do the WSS and the GDM. Any construction sloping discussed in this report are for design/planning purposes only and should not be interpreted as a direction of what will constitute safe slopes in the field during construction.

Groundwater seepage zones should be expected within the proposed excavation areas. Where groundwater seepage is encountered, erosion could occur such that the stability of temporary excavation slopes is adversely affected. The Contractor should be prepared to control groundwater seepage and prevent erosion that could cause slope instability.

Depending on the space or alternative routes available for traffic diversion around the excavation, temporary shoring walls may be required to allow for culvert excavation and staging the work. Shoring walls may also be required to protect or excavate to install utilities. The design of any temporary shoring proposed by the Contractor for this Project should be in accordance with Washington Department of Occupational Safety and Health (DOSH) and GDM guidelines. All temporary shoring designs should be submitted for review and approval by the GO and the Bridge and Structures Office.

Structural shoring is required for temporary walls that support traffic. WSS Section 2-09.3(3)D (also refers to the GDM) provides design and construction requirements for structural shoring.

7.3 DEWATERING FOR STRUCTURE EXCAVATIONS

Based on the groundwater levels measured in each boring at the time of drilling and subsequent measurements from the piezometer data logger, we anticipate that the planned culvert and wing wall foundation excavations will extend below the estimated groundwater level. Depending on location, approximately 0 to 7 feet of groundwater head may be present at the bottom of the excavation during construction.

Generally, groundwater or surface water flowing into the excavation area should be routed away from the excavation area to an appropriate location where it can be treated (if necessary) and discharged. We anticipate that creek water will need to be diverted via pipe or pumping during construction, which will reduce, although not eliminate, groundwater flow into excavations. As such, some dewatering of excavations will likely be necessary during excavation, placement, and backfilling of the culverts and retaining walls.

The use of sump pumps and gravel drainage/stabilization layers may prove feasible for dewatering excavations, particularly if downstream gravity drainage outlets are also provided. However, depending on construction staging/sequencing, the contractor may need to install well points prior to the excavation. Due to the relatively high permeability of the sandy site soils, multiple closely space well points may be needed. We therefore recommend including a Special Provision for dewatering in the contract.

Dewatering is the responsibility of the Contractor, who is solely responsible for construction means and methods and site safety. The Contractor should select, design, construct, and operate the dewatering system, in conjunction with the Contractor’s design and construction of the excavation/shoring system.

8 RECOMMENDED ADDITIONAL SERVICES

Because the future performance and integrity of the structural and geotechnical elements of this project will depend largely on proper PS&E preparation and diligent construction procedures, we recommend that the Geotechnical Office (GO) in conjunction with the Regional Materials Engineer (RME) provide the following post-report services:

- The GO should prepare the Summary of Geotechnical Conditions to be included in the PS&E as an appendix. The summary should be prepared as part of the PS&E review process.
- The GO/RME should review all construction plans and specifications to verify that the design criteria presented in this report have been interpreted correctly and properly integrated into the design.
- The GO/RME should attend pre-construction conferences with the Construction Project Engineer and Contractor to discuss important construction related issues.
- The GO/RME should review Contractor submittals for temporary shoring, permanent SE walls.
- The RME should observe the exposed subgrade for culverts and wing walls where appropriate after completion of stripping and excavation to contract elevations. The RME should confirm that suitable soil conditions have been reached and determine appropriate subgrade compaction methods.
- The RME should observe the placement of permanent drainage systems as appropriate.

In addition to the aforementioned services, the Geotechnical Office can provide inspector training for construction personnel, assist in change of conditions claims, and review cost-reduction incentive proposals (CRIPs).

9 CLOSURE

This geotechnical report was prepared to summarize our subsurface explorations, laboratory tests, and engineering analyses and to provide design recommendations and construction considerations for the culvert at the SR 539 Duffner Ditch site. This report should not be used for other purposes without contacting the WSDOT Geotechnical Office for a review of the applicability of such reuse. This report should be made available to prospective contractors for their information or factual data only and not as a warranty of ground conditions.

10 REFERENCES

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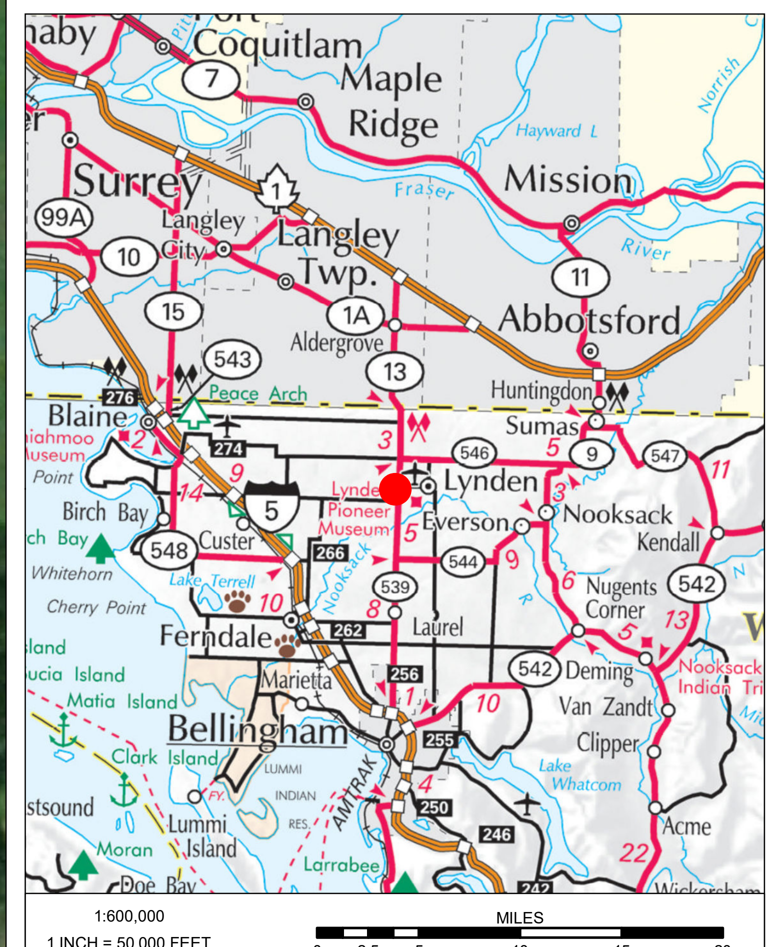
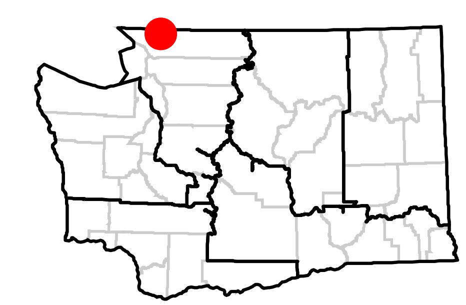


LEGEND

- Site
- Milepost - 1 Mile
- State Route
- Local Access
- WSDOT Regions
- County Boundaries (1:500K)

N



JOB # XL6478 **STATE ROUTE** 539 **MILEPOST(S)** 11.08

FIGURE 1: SITE VICINITY

SR 539 / Duffner Ditch - Bertrand Creek
Remove Fish Barrier

WSDOT GEOTECHNICAL OFFICE

PREPARED BY TropleT Date: October 7, 2021

File Location: \\wsdot.loc\ml\Group\Geotechnical\GEOGIS\Projects\SR-539\SR539-XL5949-BW-DuffnerDitchBertrandCk\Projects\SR539-XL4618-BW-DuffnerDitchBertrandCk-SiteVicinity.mxd

Legend

- Active Test Boring Locations
- Milepost - 1/10th Mile
- Milepost - 1 Mile
- State Route
- Local Access
- NHD Rivers & Streams
- Proposed Fish Passage Culvert

N

FEET
0 25 50 100 150 200
1:1,200
1 INCH = 100 FEET

JOB # XL6478 **STATE ROUTE** 539 **MILEPOST(S)** 11.08

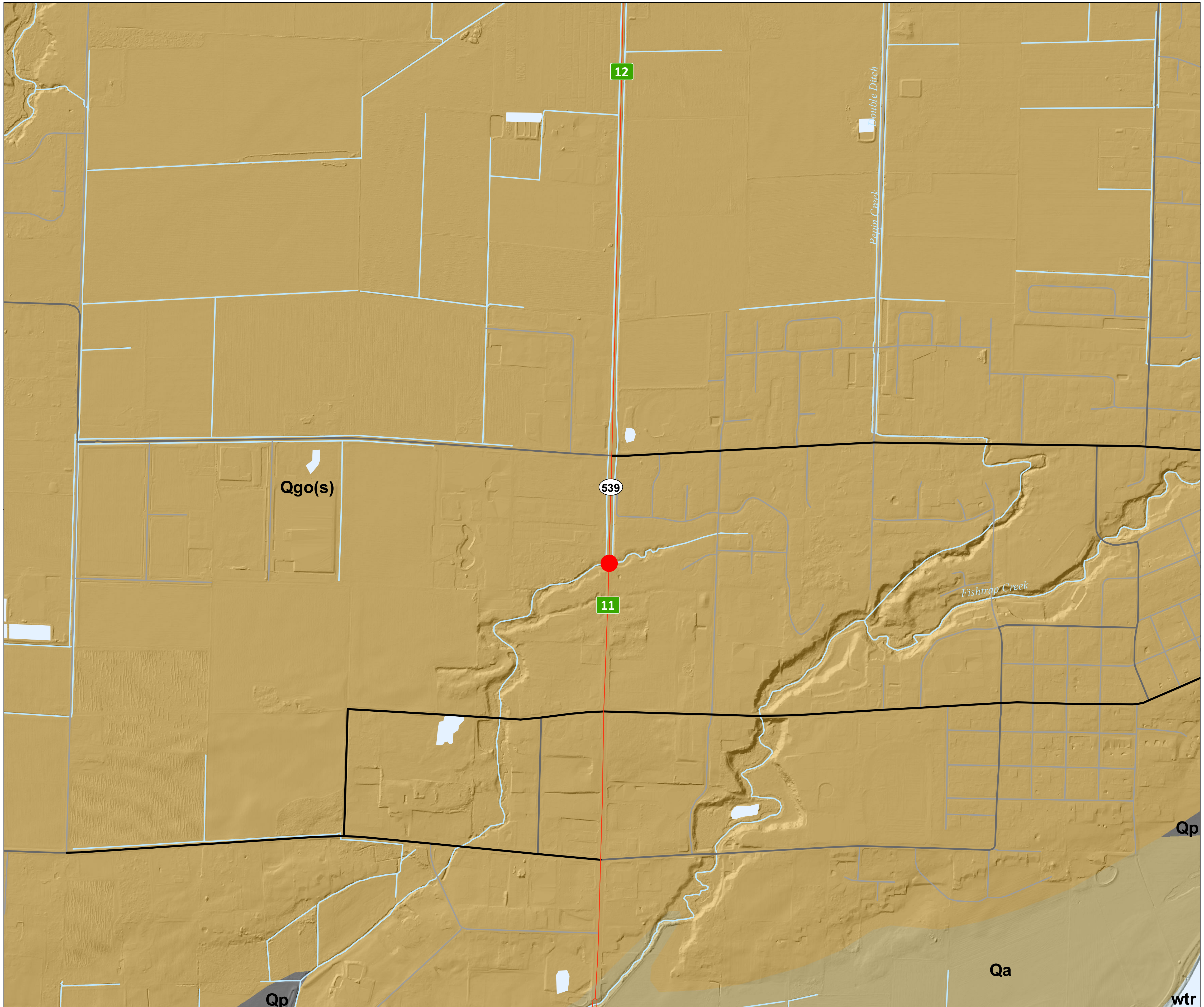
FIGURE 2: SITE EXPLORATION PLAN

SR 539 / Duffner Ditch - Bertrand Creek
Remove Fish Barrier

WSDOT GEOTECHNICAL OFFICE

PREPARED BY TropleT Date: September 1, 2022

File Location: \\mltumfl02\Group\Geotechnical\GEOGIS\Projects\SR-539\SR539-XL5949-BW-DuffnerDitchBertrandCk\Projects\SR539-XL4618-BW-DuffnerDitchBertrandCk-TestBoringLocations.mxd



Legend

- Milepost - 1/10th Mile
- Milepost - 1 Mile
- Site
- State Route
- Arterials
- Collectors
- Local Access
- NHD Waterbodies
- NHD Rivers & Streams

Geologic Units 100K

Holocene

- Qp - peat deposits

Holocene-Pleistocene

- Qa - alluvium

Pleistocene

- Qgo(s) - continental glacial outwash, Fraser-age, Sumas Stade

Other

- wtr - water

N

100K Geology: Washington State Department of Natural Resources, Division of Geology and Earth Resources, 2010.

FEET
0 250 500 1,000 1,500 2,000
1:12,000
1 INCH = 1,000 FEET

JOB # XL6478 **STATE ROUTE** 539 **MILEPOST(S)** 11.09

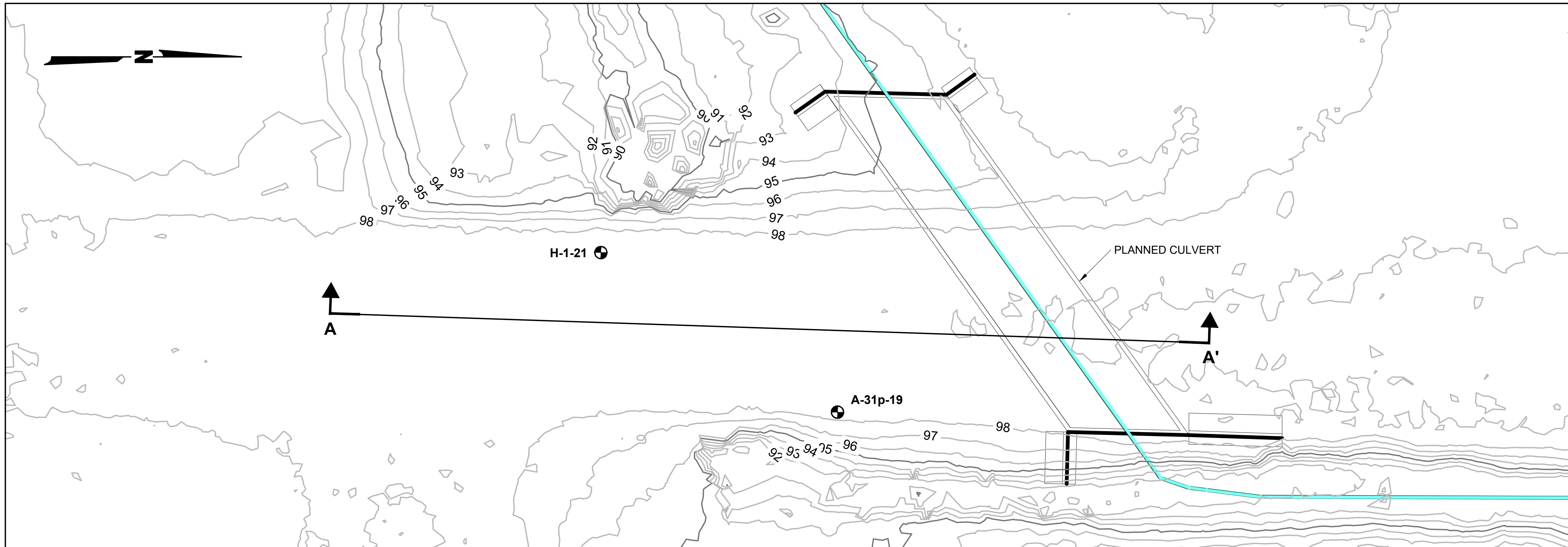
FIGURE 3: 100K GEOLOGY MAP

SR 539 / Duffner Ditch - Bertrand Creek
Remove Fish Barrier

WSDOT GEOTECHNICAL OFFICE

PREPARED BY TropleT Date: October 7, 2021

File Location: \\wsdot.loc\ml\Group\Geotechnical\GEOGIS\Projects\SR-539\SR539-XL5949-BW-DuffnerDitchBertrandCk\Projects\SR539-XL4618-BW-DuffnerDitchBertrandCk-Geology.mxd

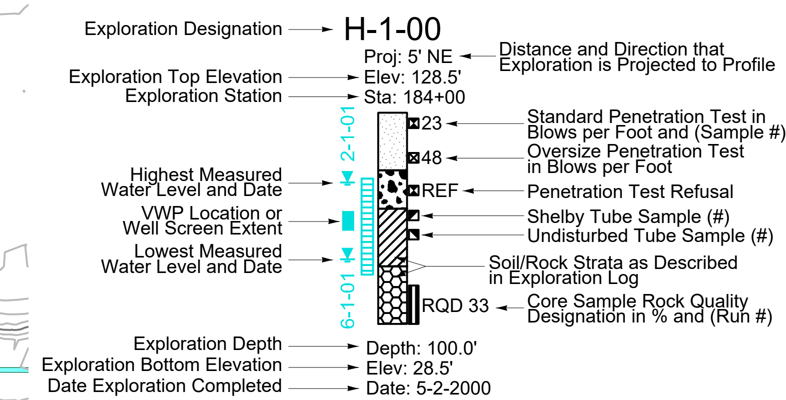


LEGEND

B-1 Boring Designation and Approximate Location

A Generalized Subsurface Profile

EXPLORATION LOG LEGEND



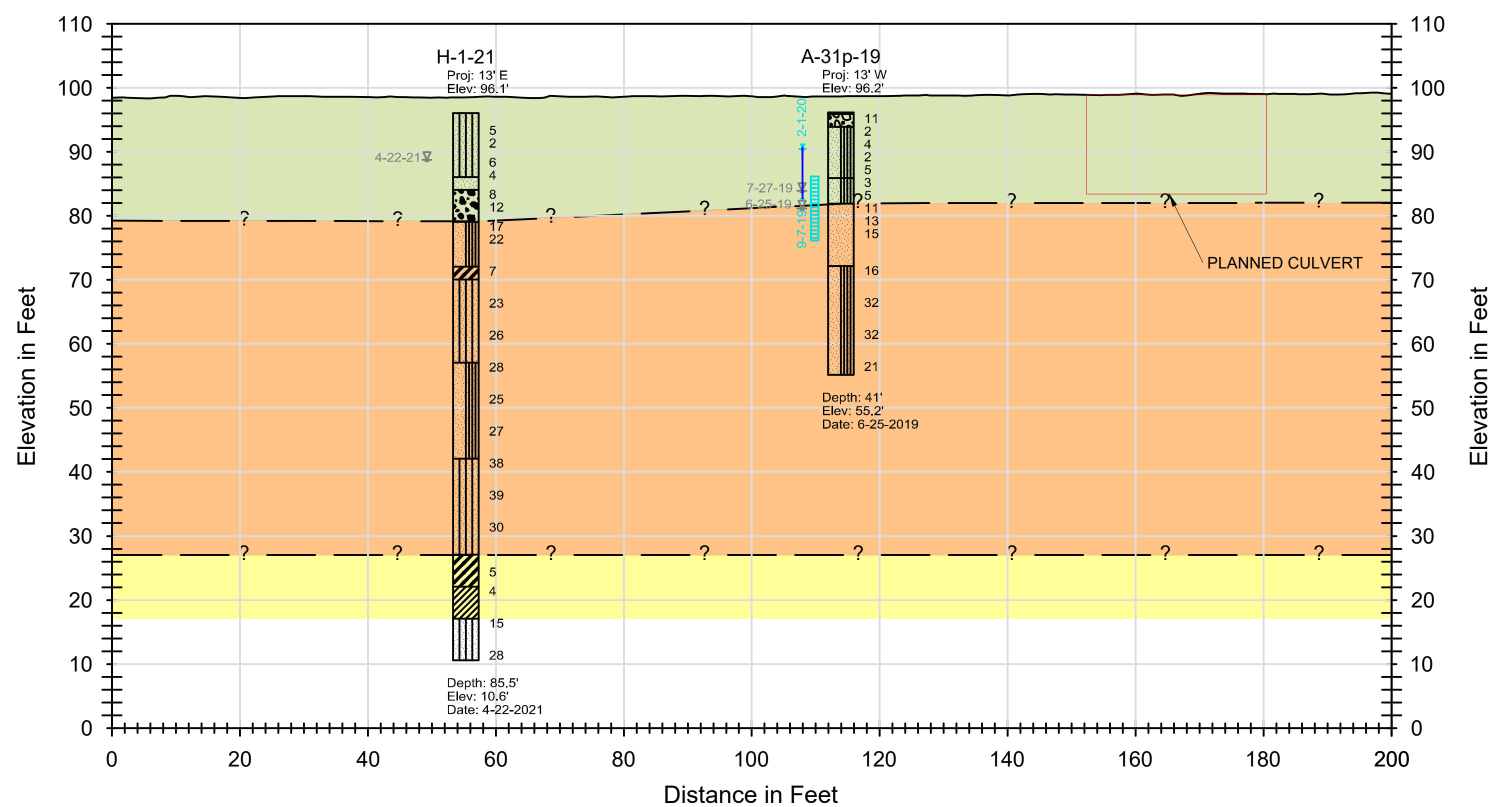
ENGINEERING STRATIGRAPHIC UNIT (ESU) COLOR LEGEND

ESU1: Alluvium: Very loose to medium dense SAND and GRAVEL.

ESU2: Outwash: Medium dense to dense SAND

ESU3: Glaciomarine Clay: Soft to medium stiff CLAY.

Recorded Seasonal High and Low Water Levels



NOTES:

1. The datum reference for this figure is: NAD 83/91 HARN, NAVD88, SPN (ft). The exploration locations were surveyed by HQ Geotech Office.
2. ESUs, if shown, are subjective and are provided for reference only.
3. Water measurements are those measured during the monitoring period (see Section 4.3.2 of the report) and could be higher or lower than shown based on seasonal or other effects.
4. Stratigraphic layer divisions, if shown, are estimated at borings. Layer divisions may vary in between or beyond borings.

0 30 60

Scale in Feet

JOB# XL6478 **STATE ROUTE** 539 **MILEPOST(S)** 11.08

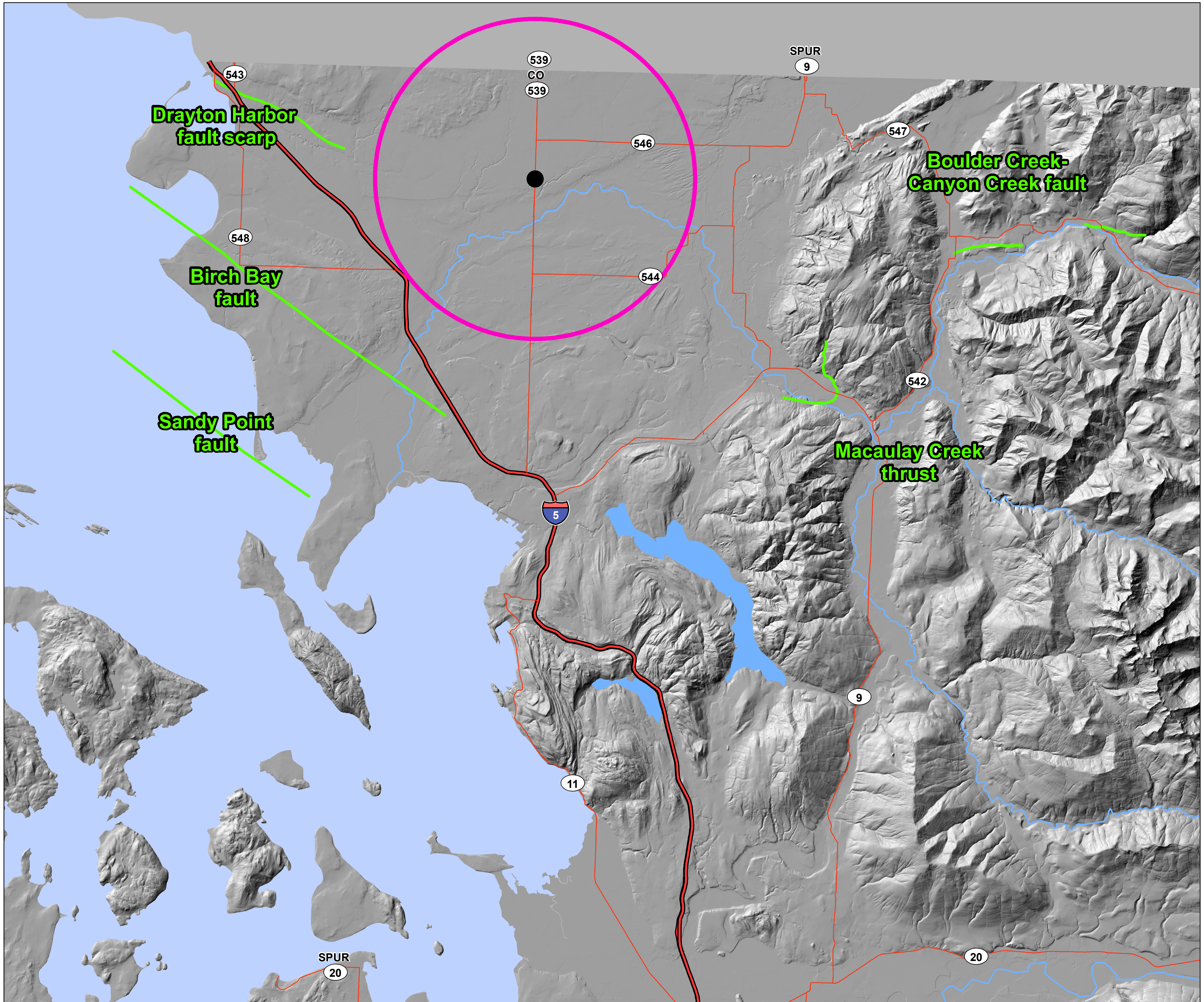
FIGURE 4: SUBSURFACE PROFILE

SR 539 / Duffner Ditch - Bertrand Creek Remove Fish Barrier

WSDOT GEOTECHNICAL OFFICE

PREPARED BY mschweitzer Date: September 2, 2022

File: \\haleyaldrich.com\share\sea_projects\Notebooks\1950112_SR 539_Duffner_Ditch_Fish_Passage_MS8328\CAD\1950112_001_0001_WSDOT_PLAN_PROFILE.dwg Layout:11x17 - H



N

Legend

- Site
- 6 Mile Buffer of Site
- U.S. Interstate
- State Route
- USGS Quaternary Faults
- Major Rivers
- Major Lakes
- Major Shorelines

Approximate Distance to Fault in Miles

Drayton Harbor fault scarp	7.2
Birch Bay fault	9.2
Macaulay Creek thrust	12.6
Sandy Point fault	14.7
Boulder Creek-Canyon Creek fault	16.1

MILES
0 1.25 2.5 5 7.5 10

1:240,000
1 INCH = 20,000 FEET

JOB # XL6478 **STATE ROUTE** 539 **MILEPOST(S)** 11.08

FIGURE 5: FAULT MAP

SR 539 / Duffner Dithc - Bertrand Creek
Remove Fish Barrier

WSDOT **GEOTECHNICAL OFFICE**

PREPARED BY TropleT Date: June 8, 2022

File Location: \\wsdot.loc\ml\Group\Geotechnical\GEOGIS\Projects\SR-539\SR539-XL5949-BW-DuffnerDitchBertrandCk\Projects\SR539-XL4618-BW-DuffnerDitchBertrandCk-Faults.mxd

APPENDIX A: FIELD EXPLORATIONS

CONTENTS

FIELD EXPLORATIONS

To characterize the surface and subsurface conditions for the project, we performed a site reconnaissance and completed two borings designated as A-31p-19 and H-1-21. The locations and elevations of the borings were determined by survey and are included on the boring logs in this appendix. The location coordinates on the boring logs are WSPN NAD83/91 coordinates. The elevations shown on the boring logs are in NAVD88.

DRILLING

All of the drilled borings were completed by Washington State Department of Transportation (WSDOT) HQ Materials Lab drill crews using a CME-45C and CME-850 truck-mounted drill rigs. The borings were completed using casing advance drilling methods. For all borings, WSDOT engineering staff supervised the field investigation effort, and field exploration staff observed the exploratory drilling, collected samples, and logged the borings.

DISTURBED SAMPLING

Disturbed samples were collected in the borings, typically at 1.5- to 5-foot depth intervals, using a standard 2-inch outer-diameter (O.D.) split-spoon sampler in conjunction with Standard Penetration Testing.

In a Standard Penetration Test (SPT) (conducted in general accordance with ASTM D1586), the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance, or N-value. In the case where the sample was driven a continuous length of 2 feet, the total number of blows (blow count) required to drive the sample in the interval from 6 to 18 inches is defined as the N-value. The uncorrected field SPT N-value provides a measure of in-situ relative density of granular soils (sand and gravel), and the consistency of fine-grained or cohesive soils (silt and clay). Refusal blow counts were determined in general accordance with ASTM D1586.

Field SPT N-values can be significantly affected by several factors, including the efficiency of the hammer, the type of sampler, the diameter of the borehole, the type of rod, and the length of rod between the hammer and the sampler. Other factors, such as disturbance of the soil at the bottom of the hole (due to caving, heave, or suction), also affect the blow counts and are more difficult to quantify. For the more quantifiable variables, such as hammer efficiency, correction factors can be applied to N-values to make them more directly comparable between borings that may have been drilled to different depths or using different equipment with different energy efficiencies. The average hammer efficiency for past energy efficiency calibrations performed on the rig were used to calculate the average hammer efficiency for the rig, which is the efficiency value presented on the boring logs.

All disturbed samples were visually classified in the field, sealed to retain moisture, and returned to our laboratory for additional examination and testing.

Undisturbed Sampling

Undisturbed samples were collected in 3-inch O.D. thin wall Shelby tubes which were pushed into the undisturbed soil at the bottoms of boreholes by directly pushing the tube with the drill rig. The soils exposed at the ends of the tubes were examined and classified in the field. After field classification, the ends of the tubes were sealed to preserve the natural moisture content of the samples. The sealed tubes were stored in the upright position and care was taken to avoid shock and vibration during their transport. The sealed tubes were returned to the WSDOT HQ Materials Laboratory and were stored in a moisture-controlled storage room until they were used for laboratory testing.

Well Installations

An open standpipe piezometer well was installed in boring A-31p-19 to observe the groundwater levels on a long-term basis. Details regarding well installation depths and screen interval is presented on the boring log attached in this appendix. The well was constructed using 1-inch PVC pipe. The annulus around the screened portion of the PVC pipe was backfilled with a sand filter pack. The annulus above the sand filter pack was backfilled with bentonite chips. Near the ground surface, the well was sealed with bentonite chips in accordance with the Department of Ecology requirements. The well was finished near the surface with open standpipe riser monuments set in concrete. The well was constructed in accordance with Washington Department of Ecology regulations.

Boring Logs

Summary logs of the borings are attached to this appendix. A two-page explanation of the symbols and terms used on the logs is also attached just prior to the logs. Note that soil descriptions and interfaces shown on the logs are interpretive, and actual changes may be gradual.

EXPLORATION LOG LEGEND

In Situ Sample and Test Symbols

Standard Penetration Test
Non-standard Penetration Test
Shelby Tube
Piston Sampler
WSDOT Undisturbed Sampler
Core Sample
Grab Sample
California Sampler
Vane Shear Test
Pressuremeter Test

Backfill and Instrument Symbols

Cement Surface Seal
Bentonite Chips
Bentonite Cement Grout (BCM)
Sand Filter Pack
Slough (Hole Collapse)
Pipe (Piezometer or Instrument) in BCM
Well Screen in Sand Filter Pack
Vibrating Wire Piezometer in BCM

Water Level Symbols

Water Level During Drilling	Transducer Depth
Water Range in Piezometer	DRY Water is Below Transducer

Laboratory Testing Codes

AL	Atterberg Limits Test
CD	Consolidated Drained Triaxial Test
CN	1-Dimensional Consolidation Test
CSS	Cyclic Simple Shear Test
CU	Consolidated Undrained Triaxial Test
DG	Degradation Test
DN	Density Test
DS	Direct Shear Test
DSS	Direct Simple Shear Test
GS	Grain Size Distribution Test
HC	Hydraulic Conductivity Test
HT	Hydrometer Test
JS	Jar Slake Test
LA	LA Abrasion Test
LOI	Loss on Ignition Test
MC	Moisture Content Test
PH	pH Test
PT	Point Load Compressive Test
RES	Resistivity Test
RS	Torsional Ring Shear Test
SG	Specific Gravity Test
SL	Slake Durability Test
UC	Unconfined Compression Test
UU	Unconlidated Undrained Triaxal Test

Soil Stratigraphy Symbols

COARSE GRAINED	FINE GRAINED & ORGANIC
GW: Well-graded Gravel	CL: Lean Clay
GP: Poorly graded Gravel	ML: Silt
GM: Silty Gravel	CH: Fat Clay
GC: Clayey Gravel	MH: Elastic Silt
SW: Well-graded Sand	OL: Organic Silt
SP: Poorly graded Sand	OH: Organic Clay
SM: Silty Sand	CL-ML: Silty Clay (dual symbol)
SC: Clayey Sand	PT: Peat or Highly Organic Soil

Soil classification is per Chapter 4.2 of the WSDOT Geotechnical Design Manual (GDM). The soil groups above contain less than 15% of other constituents. When more than 15% other constituents are observed, the soil group names are modified (e.g. Silty Gravel with Sand; Sandy, Elastic Silt with Gravel) per ASTM 2488. For dual classifications, a split symbol is used (e.g. CL-ML above). Refer to the Material Description column on the log for a complete description of the observed soil conditions.

Soil Density/Consistency

WSDOT GDM 4.2.5

COHESIONLESS SOILS		COHESIVE SOILS	
Blows/Ft	Density Term	Blows/Ft	Consistency Term
< 5	Very Loose	< 2	Very Soft
5 - 10	Loose	2 - 4	Soft
11 - 24	Medium Dense	5 - 8	Medium Stiff
25 - 50	Dense	9 - 15	Stiff
> 50	Very Dense	16 - 30	Very Stiff
		31 - 60	Hard
		> 60	Very Hard

(REF) is indicated on the log for any soil type when the penetration resistance exceeded 100 blows per foot (refusal conditions).

Soil Angularity

WSDOT GDM 4.2.4

Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges

Soil Moisture

WSDOT GDM 4.2.7

Dry	Absence of moisture, dusty, dry to touch
Moist	Damp but no visible water
Wet	Visible Free Water

Soil Structure

WSDOT GDM 4.2.8

Stratified	Alternating layers of varying material or color with layers at least 0.25 inch thick
Laminated	Alternating layers of varying material or color with layers less than 0.25 inch thick
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into smaller angular lumps which resists further breakdown
Disrupted	Soil structure is broken and mixed. Infers that material has moved substantially - landslide debris
Homogeneous	Same color and appearance throughout
Cemented	Particles are held together by a binding agent

DATUM ABBREVIATIONS: NAD = North American Datum; NAVD = North American Vertical Datum; SPN = State Plane North; SPS = State Plane South

WSDOT

LOG OF TEST BORING

Start Card	RE-17765
HOLE No.	A-31p-19
Sheet	1 of 3
Job No.	XL-5949
SR	539
Elevation	96.2 ft
Project	Advanced Work on Fish Barriers: SR-539/Lynden Cluster
Driller	Cooper, Richard
Lic#	2964
Component	
Inspector	Cooper, Richard #2964
Start	June 24, 2019
Completion	June 25, 2019
Well ID#	BBC-505
Equipment	CME 45C (9A4-7)
Station	
Offset	
Hole Dia (inches)	4
Historical SPT Efficiency	91.7%
Northing	713133.369
Easting	1243407.628
Collected by	Region Survey
Method	Casing Advancer
Lat	48.9432196
Long	-122.4852540
Datum	NAD 83/91 HARN, NAVD88, SPN (ft)
Drill Fluid	Polymer

Legend: SPT Efficiency; Field SPT (N); Moisture Content; RQD (scale 20, 40, 60, 80)

Depth (ft)	Elevation (ft)	Blows/6" (N) and/or RQD FF	Sample No. (Tube No.)	Lab Tests	Description of Material
0					ASPHALT.
	95.0	5 6 5 5 (11)	D-1		Poorly graded GRAVEL with sand, sub-rounded, medium dense, brown, moist, homogeneous. Recovered: 1.0 ft Retained: 1.0 ft
		0 1 1 1 (2)	D-2		Poorly graded SAND with silt and gravel, sub-angular, very loose, brown, wet, stratified. Recovered: 0.8 ft Retained: 0.8 ft
5		1 2 2 1 (4)	D-3	MC GS	SP-SM, MC=17% Poorly graded SAND with silt and gravel, sub-rounded, very loose, brown, wet, homogeneous. Recovered: 0.8 ft Retained: 0.8 ft
	90.0	1 1 1 1 (2)	D-4		Poorly graded SAND with silt and gravel, sub-rounded, very loose, brown, wet, homogeneous. Recovered: 0.5 ft Retained: 0.5 ft
		1 2 3 1 (5)	D-5		Poorly graded SAND with silt and gravel wood debris present, sub-rounded, loose, brown, wet, homogeneous. Recovered: 0.5 ft Retained: 0.5 ft
10	85.0	1 1 2 2 (3)	D-6		Poorly graded SAND with silt and organics, sub-rounded, very loose, brown, wet, homogeneous. Recovered: 1.0 ft Retained: 1.0 ft
		1 2 3 3 (5)	D-7	MC GS	SP-SM, MC=23% Poorly graded SAND with silt and organics, sub-rounded, loose, brown, wet, homogeneous. Recovered: 1.0 ft Retained: 1.0 ft
15		4 6 5 5 (11)	D-8		Poorly graded SAND with gravel, sub-rounded, medium dense, brown, wet, homogeneous. Recovered: 0.8 ft Retained: 0.8 ft
	80.0	4 6 7 8 (13)	D-9		Poorly graded SAND with gravel, sub-rounded, medium dense, brown, wet, homogeneous. Recovered: 1.0 ft Retained: 1.0 ft
		5 7 8 10	D-10	MC GS	SP, MC=17% Poorly graded SAND with gravel, sub-rounded, medium dense, brown, wet, homogeneous. Recovered: 1.0 ft Retained: 1.0 ft
20					

Groundwater: Groundwater readings were taken from 06-25-2019 to 08-20-2019; 07-27-2019; 06-25-2019

LOG OF TEST BORING

WSDOT

Job No. XL-5949 SR 539 Elevation 96.2 ft

HOLE No. A-31p-19

Sheet 2 of 3

Project Advanced Work on Fish Barriers: SR-539/Lynden Cluster

Driller Cooper, Richard

Legend: SPT Efficiency; Field SPT (N); Moisture Content; RQD (scale 20, 40, 60, 80)

Depth (ft)	Elevation (ft)	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material
	75	(15)				
25		5 8 8 10 (16)		D-11		Poorly graded SAND with silt, medium dense, brown, moist, homogeneous. Recovered: 1.0 ft Retained: 1.0 ft
	70					
30		10 15 17 22 (32)		D-12		Poorly graded SAND with silt, dense, brown, moist, homogeneous. Recovered: 1.2 ft Retained: 1.2 ft
	65					
35		13 15 17 14 (32)		D-13	MC GS	SP-SM, MC=25% Poorly graded SAND with silt, dense, brown, moist, homogeneous. Recovered: 1.7 ft Retained: 1.7 ft
	60					
40		3 8 13 15 (21)		D-14		Poorly graded SAND with silt, medium dense, brown, moist, homogeneous. Recovered: 2.0 ft Retained: 2.0 ft
	55					A flush mount monument was installed on this boring.
45						

Columns also include: Profile, Groundwater, Instrument.

ENTERPRISE BORING LOG XL-5949 539 LYNDEN CLUSTER.GPJ ENTERPRISE DATA TEMPLATE.GDT 9/3/19

WSDOT

LOG OF TEST BORING

Job No. XL-5949 SR 539 Elevation 96.2 ft

HOLE No. A-31p-19

Sheet 3 of 3

Project Advanced Work on Fish Barriers: SR-539/Lynden Cluster

Driller Cooper, Richard

Depth (ft)	Elevation (ft)	Profile	SPT Efficiency / Field SPT (N) / Moisture Content / RQD (20 40 60 80)	Blows/6" (N) and/or RQD FF	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	50							The implied accuracy of the borehole location information displayed on this boring log is typically sub-meter in (X,Y) when collected by the HQ Geotech Office and sub-centimeter in (X,Y,Z) when collected by the Region Survey Crew.		
50	45							End of test hole boring at 41 ft below ground elevation. This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data. Note: REF = SPT Refusal		
55	40							Bail/Recharge test: Hole Diameter: 4.0 in Depth of boring during bail test: 41 ft Depth of casing during bail test: 34 ft Water depth before bailing: 2.1 ft Bailed bore hole water level to 24.5 ft Recharge after 1 minutes: 23.9 ft Recharge after 2 minutes: 23 ft Recharge after 3 minutes: 22.3 ft Recharge after 5 minutes: 21.8 ft Recharge after 10 minutes: 19.8 ft Recharge after 15 minutes: 19.5 ft Recharge after 20 minutes: 18.3 ft Recharge after 25 minutes: 17.9 ft Recharge after 30 minutes: 17 ft Recharge after 35 minutes: 16.5 ft Recharge after 40 minutes: 16.2 ft Recharge after 45 minutes: 15.8 ft Recharge after 50 minutes: 15.5 ft Recharge after 55 minutes: 15.2 ft Recharge after 60 minutes: 15 ft		
60	35									
65	30									
70										

LOG OF BORING H-1-21

Project: SR 539/Duffner Ditch - Pre-Design		*Job Number:* MS8328	*Route & MP Range:* SR 539 MP 10.58 - 11.68
Northing: 713,075.7 feet	*Latitude:* 48.943060 deg.	*Driller/Inspector:* Henderson, Danny (#2742) / Cooper, Kerry (#2552)	
Easting: 1,243,379.7 feet	*Longitude:* -122.485365 deg.	*Start Card:* SE76475 AE64408	
Elevation: 96.1 feet	*Collector:* Region Survey	*Drilling Method:* Casing Advancer	*Hole Diam.:* 4 in
Horizontal/Vertical Datum: NAD 83 HARN, SPN / NAVD88		*Equipment:* CME 850 (ID:9C2-5)	*Rod Type:* HQ
Started: 21 April 2021	*Completed:* 22 April 2021	*Hammer Type:* Autohammer	*Historic Efficiency:* 84.5%

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Type	Sample Number	Lab Tests	Material Description	Water Observations
	95	2 2 3 (5) Rec=0.9'		D-1	GS, AL	SILTY SAND WITH GRAVEL, subangular, loose, dark brown, moist.	
5	90	2 1 1 (2) Rec=0.6'		D-2	GS, AL	SILTY SAND WITH GRAVEL, subrounded, very loose, dark brown, moist.	
		2 2 4 (6) Rec=1.0'		D-3	GS, AL	SILTY SAND, subrounded, loose, dark brown, wet, with wood.	04-22-21 HK
10	85	2 3 1 (4) Rec=0.9'		D-4	GS, AL	SILTY SAND, subangular, very loose, dark brown, wet, with wood.	
		2 4 4 (8) Rec=0.9'		D-5	GS, AL	POORLY GRADED SAND WITH GRAVEL, subrounded, loose, gray, wet.	
15	80	3 7 5 (12) Rec=0.7'		D-6	GS, AL	WELL-GRADED GRAVEL WITH SAND, subrounded, medium dense, gray, wet.	
		7 7 10 (17) Rec=1.0'		D-7		POORLY GRADED SAND WITH SILT, subrounded, medium dense, gray, wet, homogeneous.	
20	75	6 11 11 (22) Rec=1.0'		D-8	GS, AL	POORLY GRADED SAND WITH SILT, subrounded, medium dense, gray, wet, homogeneous.	
25	70	2 2 5 (7) Rec=1.5'		D-9	AL	FAT CLAY, medium stiff, gray, wet, homogeneous.	
		Rec=2.0'	P	PS-10	GS, AL	SILTY SAND, gray-brown, wet, fine sand.	
30		5		D-11	GS, AL	SILTY SAND, medium dense, gray-brown, wet, homogeneous, fine sand.	

Moisture Content (%) ✖ · Penetration Resistance (blows/ft): Field N ◆, SPT N_{60} ◇ · 0 20 40 60 80 100

CONTINUED NEXT PAGE (*see last page for notes*)

STANDARD BORING LOG MS8328_539 DUFFNERDITCH.GPJ 2020 WSDOT GINT TEMPLATE.GDT 20/10/22

Project: SR 539/Duffner Ditch - Pre-Design *Job Number:* MS8328 *Route & MP Range:* SR 539 MP 10.58 - 11.68

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description
	65	10 13 (23) Rec=1.1'	D-11	GS, AL	
35	60	9 13 13 (26) Rec=1.2'	D-12		SILTY SAND, dense, gray-brown, wet, homogeneous, fine sand.
40	55	9 14 14 (28) Rec=1.5'	D-13	GS, AL	POORLY GRADED SAND WITH SILT, dense, gray, wet, homogeneous.
45	50	6 12 13 (25) Rec=1.3'	D-14		POORLY GRADED SAND WITH SILT, dense, gray, wet, homogeneous.
50	45	6 10 17 (27) Rec=0.9'	D-15		POORLY GRADED SAND WITH SILT, dense, gray, wet, homogeneous.
55	40	10 16 22 (38) Rec=1.1'	D-16		SILTY SAND, dense, gray, wet, homogeneous.
60	35	9 19 20 (39) Rec=1.3'	D-17		SILTY SAND, dense, gray, wet, homogeneous.
65	30	13 17 13 (30) Rec=1.1'	D-18	GS, AL	SILTY SAND, dense, gray, wet, homogeneous.

Chart legend: Moisture Content (%) ✖; Penetration Resistance (blows/ft): Field N ◆, SPT N_{60} ◇; scale 0 20 40 60 80 100

STANDARD BORING LOG MS8328_539 DUFFNERDITCH.GPJ 2020 WSDOT GINT TEMPLATE.GDT 20/10/22

WSDOT

LOG OF BORING H-1-21

Project: SR 539/Duffner Ditch - Pre-Design *Job Number:* MS8328 *Route & MP Range:* SR 539 MP 10.58 - 11.68

Depth (feet)	Elevation (feet)	Blows/6" (N bpf) and other Field Data	Sample Number	Lab Tests	Material Description
70	25	Rec=2.0'	PS-19		FAT CLAY, medium stiff, gray, wet.
		2 2 3 (5) Rec=1.5'	D-20		
75	20	2 2 2 (4) Rec=0.7'	D-21	GS, AL	LEAN CLAY WITH SAND, soft, gray, wet, homogeneous.
80	15	2 3 12 (15) Rec=1.4'	D-22	GS, AL	SILTY SAND, medium dense, gray, wet, homogeneous.
85		6 12 16 (28) Rec=1.2'	D-23		SILTY SAND, medium dense, gray, wet, homogeneous.

Moisture Content (%) / Penetration Resistance (blows/ft): Field N, SPT N_{60} (scale 0 20 40 60 80 100)

HOLE ENDED AT 85.5 FEET ON 4-22-2021

NOTES:

1. This is a summary log of the boring. Soil/rock descriptions are derived from visual field identifications and laboratory test data (where tested). See exploration log legend for explanation of graphics and abbreviations.
2. The implied accuracy of the location information displayed on this log is typically sub-meter (X,Y) when collected using GPS methods by the Geotechnical Office and sub-centimeter (X,Y,Z) when collected by the Region survey crew.
3. Where oversized samplers were used, a correction was made to the N-value per the AASHTO Manual on Subsurface Investigations, 1988.
4. The groundwater level(s), if shown, represents observations made during drilling. The groundwater level should be considered approximate and will vary based on seasonal and other effects.

APPENDIX B: LABORATORY TEST RESULTS

CONTENTS

LABORATORY TEST RESULTS

The soil samples were classified visually in the field in general accordance with Chapter 4 of the GDM. The classification criteria in the GDM is a modified version of ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Once transported to the laboratory, the samples were re-examined, various laboratory tests were performed, and the field classifications were modified accordingly. We refined our visual-manual soil classifications based on the results of the laboratory tests, using the Standard Practice for Classification of Soils for Engineering Purposes (ASTM D2487).

Note that ASTM D2487 requires Cc and Cu values to classify sands that have 12 percent or less passing the U.S. No. 200 mesh sieve. If a tested sample specimen had greater than 10 percent but less than or equal to 12% passing the U.S. No. 200 mesh sieve, then the value of D10 was assumed to be 0.07 millimeters. The relative density modifiers for the sample descriptions shown on the boring logs are based on the original field blow counts and not on adjusted blow count values.

MOISTURE (NATURAL WATER) CONTENT

Natural moisture content determinations were performed in accordance with ASTM D2216 on selected soil samples. The natural moisture content is a measure of the amount of moisture in the soil at the time the explorations are performed and is defined as the ratio of the weight of water to the dry weight of the soil, expressed as a percentage. The results of the moisture content determinations are shown on the Laboratory Summary sheets attached at the end of this appendix and are included on the boring logs in Appendix B.

ATTERBERG LIMITS

Atterberg limits were determined on selected samples in accordance with AASHTO T89 and AASHTO T90. This analysis yields index parameters of the soil that are useful in soil classification and analyses, including liquefaction analysis. An Atterberg limit test determines a soil’s liquid limit (LL) and plastic limit (PL). These are the maximum and minimum moisture contents at which the soil exhibits plastic behavior. A soil’s plasticity index (PI) can be determined by subtracting PL from LL. The test results are shown on the Laboratory Summary sheets attached at the end of this appendix and are included on the boring logs in Appendix B.

PARTICLE-SIZE ANALYSES

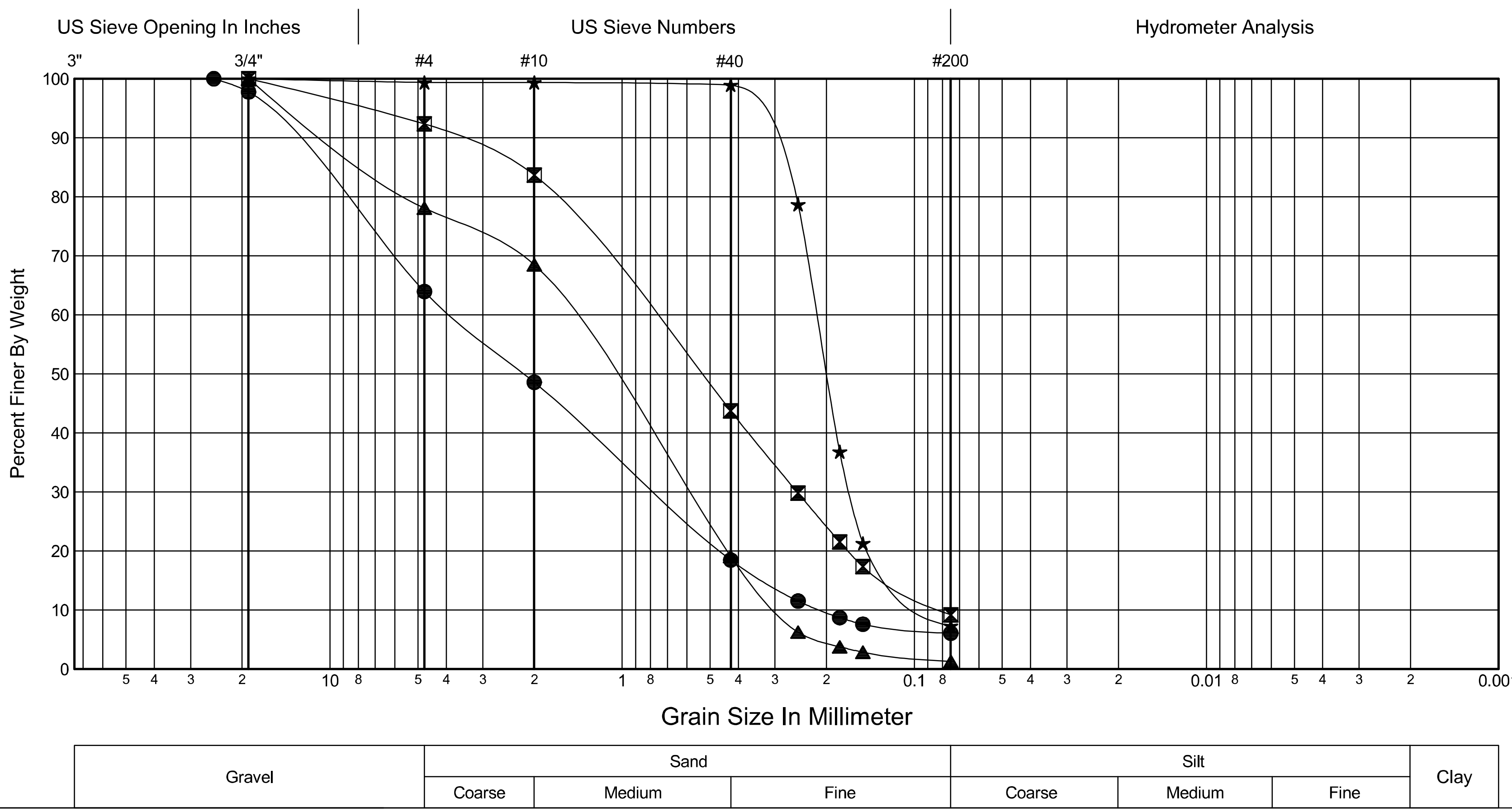
Particle-size analyses were conducted on selected samples to determine their grain-size distributions. Grain-size distributions were determined by sieve analysis in general accordance with AASHTO T27-11. For selected samples, the material retained on the U.S. No. 200 mesh sieve was shaken through a series of sieves to determine the distribution of the plus No. 200 fraction. For some tests, only the percentage of the sample passing the U.S. No. 200 (0.075mm) mesh sieve was determined. Several grain-size distributions also included a hydrometer analysis by AASHTO T88. The hydrometer analysis yields the grain-size distribution of the sample fraction finer than the U.S. No. 200 mesh sieve. Results of the particle-size analysis are shown on the Laboratory Summary sheets that are attached at the end of this appendix.

Job No. **XL-5949** Date **July 29, 2019**
Hole No. **A-31p-19** Sheet **1**
Project **Advanced Work on Fish Barriers: SR-539/Lynden Cluster**

Laboratory Summary

WSDOT

	Depth (ft)	Sample No.	USCS	Description	MC%	LL	PL	PI	Moist Density (lbs/ft^3)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	Cc	Cu	D60	D50	D30	D20	D10
●	4.5	D-3	SP-SM	POORLY GRADED SAND with SILT and GRAVEL	17						36.1	57.9	6.1	0.7	18.2	3.805	2.17	0.77	0.46	0.210
⊠	12.5	D-7	SP-SM	POORLY GRADED SAND with SILT and Organics	23						7.7	83.2	9.2	1.0	9.9	0.799	0.54	0.25	0.17	0.080
▲	18.5	D-10	SP	POORLY GRADED SAND with GRAVEL	17						21.9	76.8	1.3	0.8	5.2	1.533	1.12	0.60	0.44	0.292
★	34.0	D-13	SP-SM	POORLY GRADED SAND with SILT	25						0.6	92.1	7.3	1.5	2.5	0.216	0.20	0.17	0.14	0.086

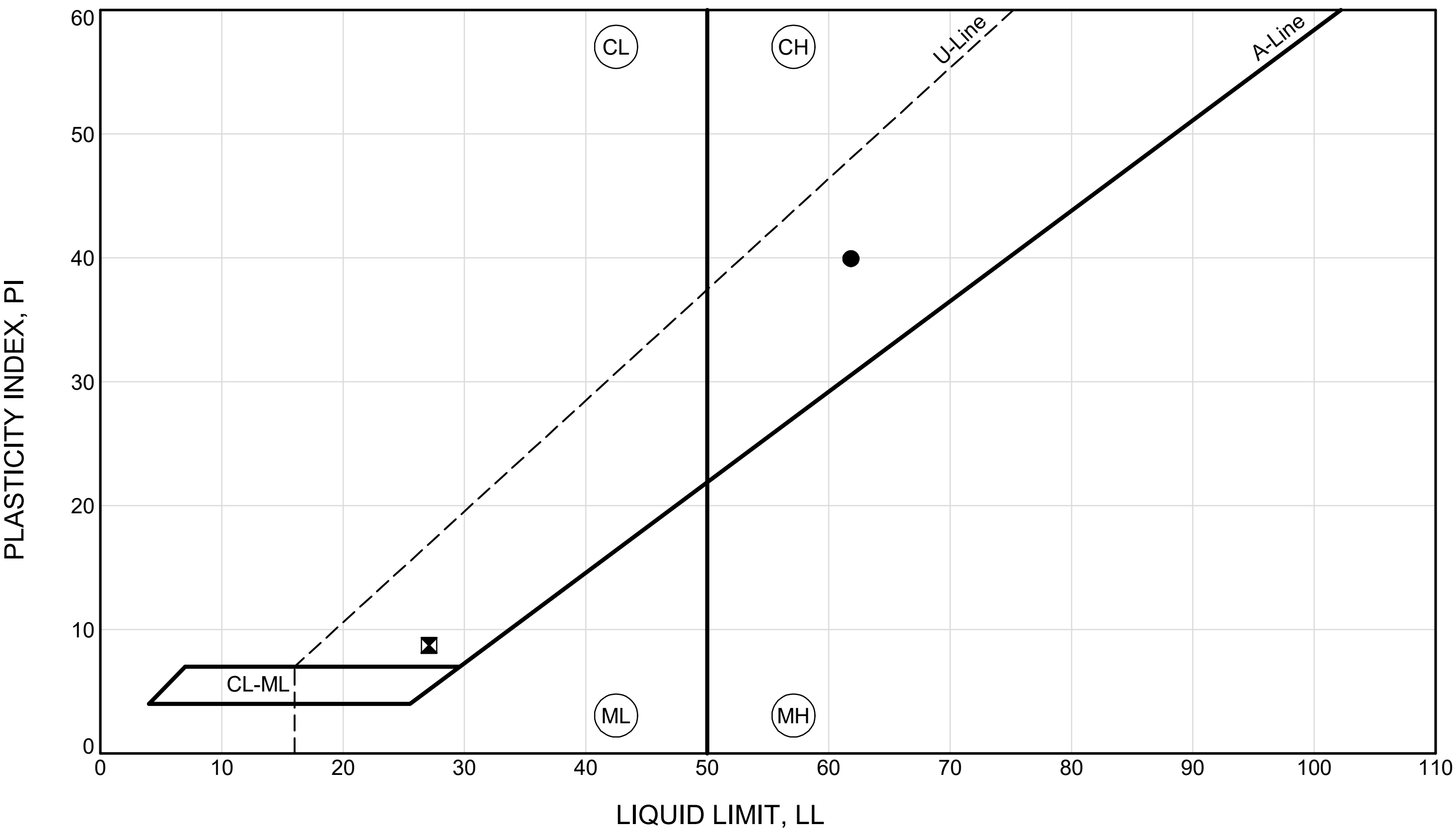


XL-5949 539 LYNDEN CLUSTER.GPJ PUBLIC-COMMON LIBRARY.GLB 7/29/19

WSDOT

ATTERBERG LIMITS SUMMARY

Job No: **MS8328**
Project: **SR 539/Duffner Ditch - Pre-Design**



Symbol	Hole No.	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Fines (%)	Silt (%)	Clay (%)
●	H-1-21	24.0	D-9		FAT CLAY	5-10-21	48	62	22	40			
⊠	H-1-21	74.0	D-21	CL	LEAN CLAY with SAND	5-10-21	29	27	18	9	78.9		

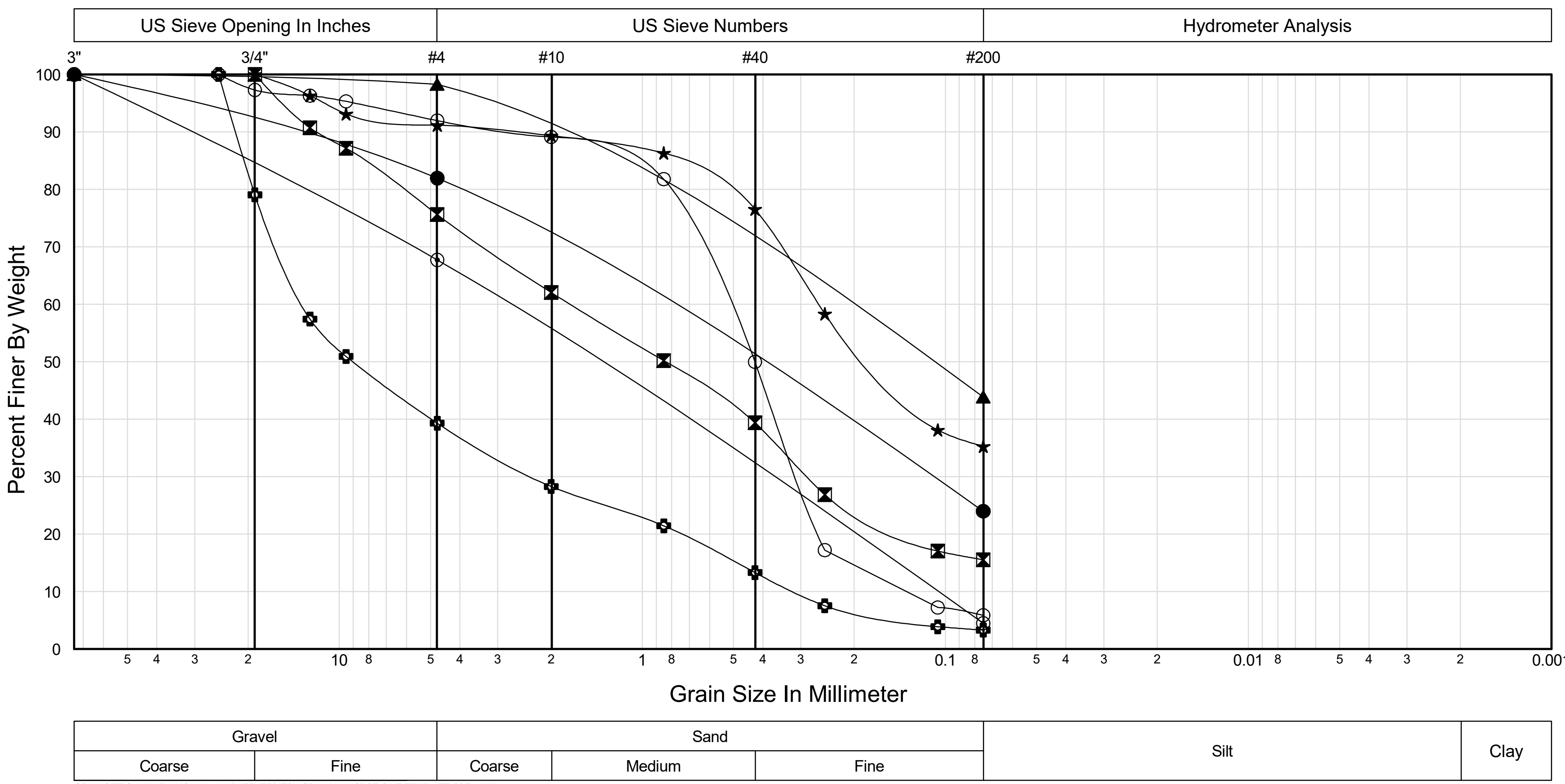
ATTERBERG SUMMARY MS8328_539 DUFFNERDITCH.GPJ 2020 WSDOT GINT TEMPLATE.GDT 6/9/21

ABBREVIATIONS:
LL = liquid limit; MC = moisture content; n/a = test attempted; NP = nonplastic; PI = plasticity index; PL = plastic limit; USCS = Unified Soil Classification System code
USCS codes listed on graph: CL = lean clay; CH = fat clay; ML = silt; MH = elastic silt; CL-ML = silty clay

GRAIN SIZE ANALYSIS SUMMARY FOR H-1-21

Job No: **MS8328**
Project: **SR 539/Duffner Ditch - Pre-Design**

Symbol	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Moist Density (lbs/ft³)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	C_c	C_u	D_{60} (mm)	D_{50} (mm)	D_{30} (mm)	D_{20} (mm)	D_{10} (mm)
●	2.0	D-1	SM	SILTY SAND with GRAVEL	5-10-21	20	n/a	n/a	NP			18.1	58.0	24.0			0.988	0.483	0.115		
⧗	4.0	D-2	SM	SILTY SAND with GRAVEL	5-10-21	18	n/a	n/a	NP			24.4	60.1	15.5			1.725	0.839	0.286	0.137	
▲	7.0	D-3	SM	SILTY SAND	5-10-21	38	n/a	n/a	NP			1.7	54.4	43.9			0.257	0.120			
★	9.0	D-4	SM	SILTY SAND	5-10-21	49	n/a	n/a	NP			8.8	55.9	35.3			0.262	0.176			
⊙	12.0	D-5	SP	POORLY GRADED SAND with GRAVEL	5-10-21	16	n/a	n/a	NP			32.3	63.2	4.5	0.5	27	2.863	1.486	0.400	0.208	0.108
✿	14.0	D-6	GW	WELL-GRADED GRAVEL with SAND	5-10-21	8	n/a	n/a	NP			60.7	36.0	3.3	1.3	42	13.143	8.991	2.290	0.751	0.314
○	19.0	D-8	SP-SM	POORLY GRADED SAND with SILT	5-10-21	19	n/a	n/a	NP			8.0	86.1	5.9	1.3	4	0.529	0.425	0.308	0.262	0.135

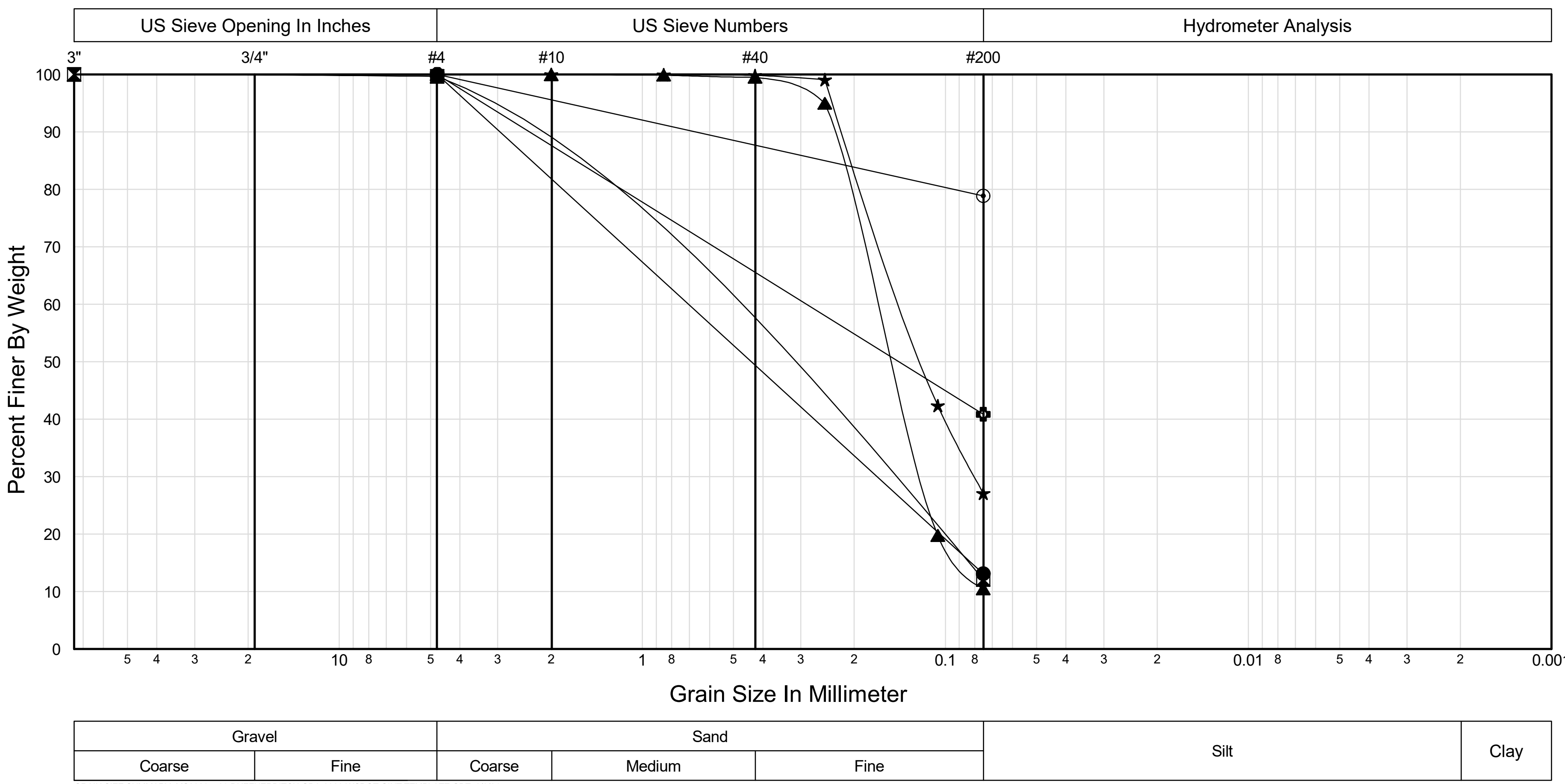


WSDOT

GRAIN SIZE ANALYSIS SUMMARY FOR H-1-21

Job No: **MS8328**
Project: **SR 539/Duffner Ditch - Pre-Design**

Symbol	Depth (feet)	Sample No.	USCS	Description	Test Date	MC (%)	LL	PL	PI	Moist Density (lbs/ft^3)	Specific Gravity	Gravel (%)	Sand (%)	Fines (%)	C_c	C_u	D_{60} (mm)	D_{50} (mm)	D_{30} (mm)	D_{20} (mm)	D_{10} (mm)
●	26.0	PS-10	SM	SILTY SAND	5-10-21	26	n/a	n/a	NP			0.0	86.9	13.1			0.704	0.437	0.168	0.104	
⧗	29.0	D-11	SM	SILTY SAND	5-10-21	26	n/a	n/a	NP			0.4	87.5	12.1	0.6	11	0.726	0.452	0.175	0.109	
▲	39.0	D-13	SP-SM	POORLY GRADED SAND with SILT	5-10-21	29	n/a	n/a	NP			0.0	89.5	10.5	1.1	2	0.168	0.150	0.119	0.106	
★	64.0	D-18	SM	SILTY SAND	5-10-21	28	n/a	n/a	NP			0.0	72.9	27.1			0.138	0.119	0.080		
⊙	74.0	D-21	CL	LEAN CLAY with SAND	5-10-21	29	27	18	9			0.0	21.1	78.9							
✤	79.0	D-22	SM	SILTY SAND	5-10-21	29	n/a	n/a	NP			0.0	59.1	40.9			0.287	0.142			



APPENDIX C: GROUNDWATER MONITORING RESULTS

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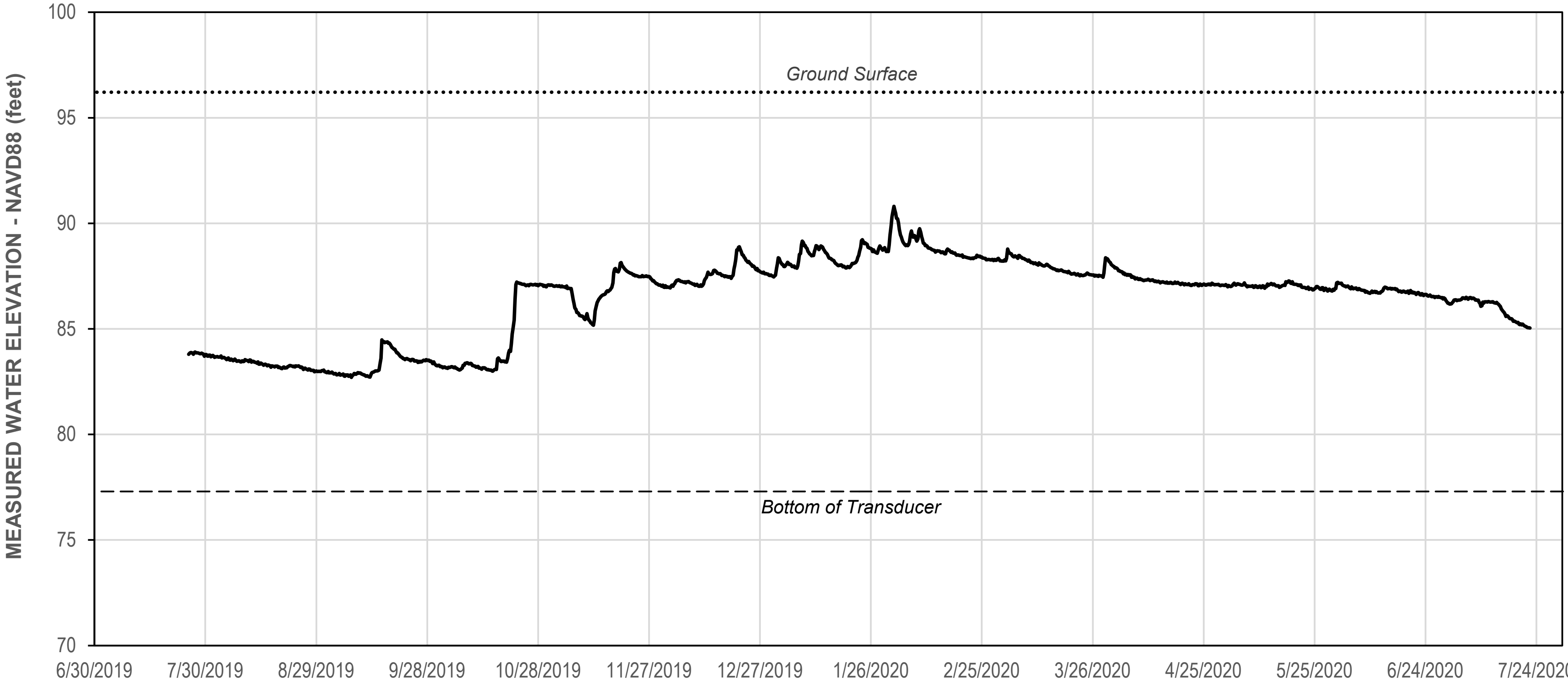

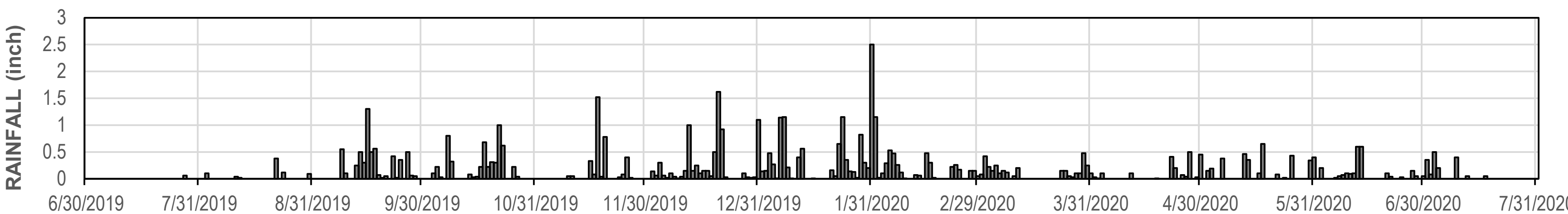

Exploration Information	
Northing (feet)	713,133
Easting (feet)	1,243,408
Ground Elevation (feet)	96
Total Boring Depth (feet)	41
Date Completed	6/25/2019

Piezo Information	Depth*	Elevation*
Piezometer Type	1-inch-diameter PVC casing	
Screened Interval	10 to 20	86.2 to 76.2
In-Situ Soil/Rock	Poorly-Graded SAND	
Highest Reading	5.4	90.8
Lowest Reading	13.5	82.7

** all units in feet*

NOTE:
Rainfall data was downloaded from https://www.ncdc.noaa.gov for the Clearbrook Station (ID USC00451484), located about 7.5 miles east of the project site.

JOB# XL5949 **STATE ROUTE 539** **MILEPOST(S) 11.08**

GROUNDWATER MEASUREMENT PLOT
BORING A-31P-20

TRIBUTARIES TO FOUR MILE CREEK
DUFFNER DITCH TO BERTRAND CREEK

WSDOT **GEOTECHNICAL OFFICE**

PREPARED BY V. Jackman **DATE: 6/8/2022**